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RENOVATING SECONDARY SEWAGE BY GROUND WATER RECHARGE WITH INFILTRATION BASINS

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RENOVATING SECONDARY SEWAGE BY GROUND WATER RECHARGE WITH INFILTRATION BASINS,

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A field project demonstrated the feasibility of renovating secondary sewage effluent by ground water recharge with infiltration basins. Maximum loading rates were obtained with cycles of 20 days flooding rotated with dry periods of 10 days in the summer and 20 days in winter. With these schedules the system could infiltrate 300-400 ft/year using a water depth of 1 ft. Grassed basins had higher infiltration rates, and a gravel covered basin had a lower infiltration rate than a bare soil basin. Essentially complete removal of BOD and fecal coliform, and significant removal of phosphorus, nitrogen and fluoride were obtained. Hydraulic properties of the aquifer were evaluated by analog from the response of piezometric heads in the ground-water system to infiltration. These properties were then used in the design of a prototype system, which would yield renovated water at an estimated total cost of about $5 per acre-foot at the pump. (Lowry-Texas)

*Water reuse, *Tertiary treatment, *Ground water recharge, Design criteria, *Infiltration, Water spreading, Cost analysis, Water table, Nitrogen, Denitrification, Phosphates, Fluorides, Boron, Coliforms, Waste water treatment

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RENOVATING SECONDARY SEWAGE BY GROUND WATER RECHARGE WITH INFILTRATION BASINS

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OFFICE OF RESEARCH AND MONITORING

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ABSTRACT

The feasibility of renovating secondary sewage effluent by ground water recharge was studied with six infiltration basins in the loamy sand of the Salt River bed. The ground water was at a depth of 10 ft and observation wells for sampling renovated water were installed inside and outside the basin area.

Infiltration rates generally ranged between 2 and 3 ft/day at 1-ft water depth. They were highest in grass-covered basins and lowest in a gravel-covered basin. Flooding periods of 2 to 3 weeks alternated with dry ups of 10 days in the summer and 20 days in the winter yielded maximum long-term infiltration, i. e., about 400 ft/year. Directional hydraulic conductivities of the aquifer were evaluated by resistance network analog and field tests. The resulting values were used to predict water table profiles and underground detention times in an operational system which would produce renovated water at about $5/acre-foot.

Suspended solids, BOD, and fecal coliform were essentially completely removed as the water seeped through the soil. With proper inundation scheduling, significant removal of nitrogen could also be obtained, especially below vegetated basins. Most of the phosphates and fluorides were also removed from the water. Boron was not removed.

CONTENTS

Section		Page
I	CONCLUSIONS	1
II	RECOMMENDATIONS	3
III	INTRODUCTION	5
IV	DESCRIPTION OF PROJECT	7
V	INFILTRATION STUDIES	13
	1. Basin management and infiltration rates	13
	2. Pressure-head and water-content measurements	25
	3. Basin management for maximum hydraulic loading	29
VI	AQUIFER STUDIES	37
	1. Hydraulic conductivity of aquifer and flow system	37
	2. Underground detention times	42
	3. Water table response	45
	4. Effect of recharge on hydraulic conductivity of aquifer	47
VII	WATER QUALITY STUDIES	49
	1. Biochemical and chemical oxygen demand	49
	2. Nitrogen	52
	3. Phosphate	63
	4. Boron	65
	5. Fluoride	65
	6. Total dissolved salts	68
	7. pH	70
	8. Coliform and soil bacteria	70
	9. Effect of grass filtration on effluent quality	75
	10. Water temperatures	78

Section		Page
VIII	DESIGN AND OPERATION OF LARGE-SCALE SYSTEM	79
	1. General design aspects	79
	2. Underground flow system	81
	3. Economic aspects	92
	4. Future projects	93
IX	ACKNOWLEDGMENTS	97
X	REFERENCES	99
XI	PUBLICATIONS	101

FIGURES

		PAGE
1	PLAN OF FLUSHING MEADOWS PROJECT	9
2	SCHEMATIC OF INFLOW AND OUTFLOW STRUCTURES FOR INFILTRATION BASINS	10
3	INFILTRATION RATES FOR RECHARGE BASINS IN 1967	14
4	INFILTRATION RATES FOR RECHARGE BASINS IN 1968	16
5	DIAGRAM OF SERIES FLOW WITH MEASURING AND SAMPLING POINTS	19
6	INFILTRATION RATES FOR RECHARGE BASINS IN 1969	21
7	ACCUMULATED INFILTRATION FOR RECHARGE BASINS IN 1969	24
8	PRESSURE HEADS OF WATER IN SOIL BENEATH BASIN 5 DURING FLOODING AND DRY UP	26
9	PRESSURE HEADS OF WATER IN SOIL BENEATH BASIN 6 DURING FLOODING AND DRY UP	27
10	INFILTRATION RATE AND K_u AT DIFFERENT DEPTHS BELOW BASIN 2	28
11	K_u AS A FUNCTION OF SOIL-WATER PRESSURE-HEAD FOR SOIL IN BASIN 2	30
12	RELATION BETWEEN WATER CONTENT AND PRESSURE HEAD FOR SOIL IN BASIN 5	31
13	SCHEMATIC PRESENTATION OF INFILTRATION DECREASE DURING INUNDATION AND RECOVERY DURING DRY UP	32
14	LONG-TERM INFILTRATION RATE IN RELATION TO LENGTH OF INUNDATION PERIODS	33
15	RESPONSE OF WATER LEVEL IN ECW TO INFILTRATION	38
16	GROUND WATER FLOW SYSTEM AT STEADY-STATE DURING RECHARGE	41
17	WATER-LEVEL PROFILES IN OBSERVATION WELLS DURING INUNDATION AND DRY UP	46
18	COD OF SECONDARY EFFLUENT AND OF RENOVATED WATER FROM EAST CENTER WELL IN 1968	50

		PAGE
19	COD OF SECONDARY EFFLUENT AND OF RENOVATED WATER FROM ECW, WELL 1-2, AND WELL 5-6 IN 1969	51
20	TOTAL NITROGEN OF SECONDARY EFFLUENT, AND NITRATE N AND AMMONIUM N IN RENOVATED WATER FROM ECW IN 1968	54
21	TOTAL NITROGEN OF SECONDARY EFFLUENT, AND NITRATE N AND AMMONIUM N IN RENOVATED WATER FROM ECW IN 1969	55
22	NITRATE N IN RENOVATED WATER FROM WELLS 1-2, ECW, AND 5-6 IN 1968	58
23	NITRATE N IN RENOVATED WATER FROM WELLS 1-2, ECW, AND 5-6 IN 1969	59
24	COLIFORM BACTERIA IN RENOVATED WATER FROM ECW IN 1968	73
25	COLIFORMS AND FECAL STREPTOCOCCI IN RENOVATED WATER FROM ECW IN 1969	74
26	PLAN AND CROSS-SECTION OF TWO PARALLEL STRIPS WITH RECHARGE BASINS WITH WELLS IN CENTER FOR COLLECTING RENOVATED WATER	80
27	GROUND WATER MOUND OF FLOW SYSTEM IN FIGURE 16 WITH VERTICAL SCALE EXAGGERATED	82
28	LINEAR FLOW BENEATH RECHARGE STRIP (ABCD) AND NODE ARRANGEMENT IN PORTION OF FLOW SYSTEM (ABEF) ANALYZED BY RESISTANCE NETWORK ANALOG	85
29	DIMENSIONLESS GRAPH OF $2 WS\bar{I}/T_e \Delta H_{A-F}$	87
30	DIMENSIONLESS GRAPH OF $\Delta H_{B-F}/\Delta H_{A-F}$ VERSUS L/S	88
31	STREAMLINES AND EQUIPOTENTIALS (IN FEET ABOVE WATER TABLE ADJACENT TO WELL) FOR HYPOTHETICAL SYSTEM IN SALT RIVER BED	89
32	EQUIPOTENTIALS (AB = 100, F = 0) IN REGION ABEF FOR DIFFERENT VALUES OF L/S	90
33	MODEL OF RIO SALADO PROJECT, SHOWING SEWAGE WATER RENOVATION SYSTEM (FRONT) AND RECREATIONAL LAKES (BACK).	95

TABLES

No.		Page
1	Driller's Log for East Well and West Center Well	8
2	Effect of Surface Condition on Infiltration Rate (1968)	18
3	Results of Hydraulic Conductivity Measurements	40
4	Occurrence of Distinct Nitrate Peaks in ECW in Relation to Start of Inundation Period and Average Infiltration Rate	44
5	COD in Mg/Liter for Various Wells	53
6	Nitrate Nitrogen Concentrations in Mg N per Liter for Various Wells	61
7	Ammonium Nitrogen Concentrations in Mg N per Liter for Various Wells	62
8	Phosphate-Phosphorus Concentrations in Mg P per Liter of Effluent, Renovated Water, and Native Ground Water	64
9	Boron Concentration in Mg B per Liter for Effluent, Renovated Water, and Native Ground Water	66
10	Fluoride Concentration in Mg F per Liter for Effluent, Renovated Water, and Native Ground Water	67
11	Total Salt Concentration in Effluent and in Water from Wells, Mg/Liter	69
12	pH of Effluent and Water from Wells (1969)	71
13	Aerobic Bacteria Counts in Recharge Basin Soil	72
14	Effect of Flow Through Basins on COD, NH_4, and NO_3 Content of Effluent (1968)	76
15	Effect of Flow Through Grass Basins and Grass Mat on COD, NH_4 and NO_3 of Effluent	77

SECTION I

CONCLUSIONS

1. The pilot project has shown that secondary sewage effluent can be effectively renovated for unrestricted irrigation and primary-contact recreation by ground water recharge with infiltration basins in the Salt River bed. In an operational system, the basins would be located on both sides of the river bed and the renovated water would be pumped from wells in the center of the river bed. Total cost of renovating sewage effluent in this manner is estimated at $5/acre-foot at the well.

2. An accumulated infiltration of 400 ft/yr can be obtained with flooding periods of 2 to 3 weeks alternated with dry-up periods of 10 days in the summer and 20 days in the winter. Thus, 1 acre of recharge basin can handle about 0.35 mgd. Grass covered basins had the highest infiltration rates. However, water depths are restricted in grass covered basins and short, frequent floodings must be employed in the spring and early summer to get a tall, dense stand of the grass.

3. The response of the ground water table beneath the infiltration basins enabled the evaluation of the horizontal and vertical hydraulic conductivity of the aquifer. These hydraulic conductivities were used in the design of an operational system to predict underground detention times and water table profiles for various geometries of recharge basins and wells for pumping renovated water.

4. Short, frequent inundations of the basins (2-4 days wet, and 3-5 days dry, for example) yielded almost complete conversion of the nitrogen in the effluent water to nitrate in the renovated water. With 2-3 week floodings alternated by 10-20 day dry ups, however, 50 to 80% of the nitrogen was removed following the passage of nitrate peaks. These peaks occurred shortly after the start of a new inundation period due to leaching of nitrified effluent from the soil held as capillary water during the preceding dry up. Nitrogen removal was greater below vegetated basins than below nonvegetated basins.

5. Prolonged operation of the infiltration basins for nitrogen removal eventually caused an increase in the ammonium level of the renovated water. Thus, short, frequent inundations, preferably in combination with growing a crop, should be used every other year or so to restore the nitrogen-removing capability of the recharge system.

6. The soil filtration process yielded essentially complete removal of suspended solids, BOD, and fecal coliform. Significant removal of phosphates and fluorides also took place. Boron, however, was not removed.

7. There was no indication of gradual clogging of the soil profile or the aquifer, or of a decrease in the renovation efficiency of the system. Thus, a ground water recharge system for renovating secondary sewage effluent should have a long, useful life.

SECTION II.

RECOMMENDATIONS

Because of the favorable results of the experimental project, larger projects of an operational nature can be started with confidence. The renovated water could, for example, be used for unrestricted irrigation and recreational lakes. The design procedures presented in this report could be used to determine the most favorable layout of recharge basins and wells, and to predict the water table positions, pumping lifts, and underground detention times for the system.

The infiltration basins for an operational system are likely to be from several acres to 10 acres or more, which is much larger than the long, narrow basins of the experimental project. Field investigations should therefore be carried out to determine if the large basins have restricted escape of air from the soil when a new inundation is started. The resulting buildup of air pressure beneath the advancing wet front in the soil can then reduce infiltration rates. Slow filling of the basin from one side to obtain a low rate of advance of the water over the bottom of the basin, and installation of air relief pipes in the soil may be effective in minimizing the buildup of air pressure in the soil when the soil is wetted.

To avoid spread of renovated sewage water into the ground water basin of the Salt River Valley, all the effluent infiltrated in the basins should be pumped out of the aquifer as renovated water. This makes it possible to determine how many pounds of nitrogen, oxygen demand, organic carbon, phosphorus, etc., are removed by the system. When a larger system is installed, the important quality parameters should be monitored, as well as the ground water levels along the periphery of the system to make sure that no renovated water spreads into the rest of the ground water basin. Underground detention times and water table profiles should be measured and compared with the predicted values to check the validity of the design procedures.

Operation of the pilot project should be continued to permit more detailed studies of the nitrogen and coliform behavior. Also, the effectiveness of various types of vegetation on nitrogen removal should be studied. Soil clogging, particularly of the surface layer, should be examined in more detail, as well as the effect of rainfall during dry up on the recovery of the infiltration. The suitability of various crops for stimulating denitrification, and the possibility of the addition of artificial carbon sources to obtain more denitrification should be investigated. Some of these studies can best be carried out in the laboratory on soil columns. Because of the interest in using renovated sewage water for recreation, the biostimulation of such water in impoundments should be studied so that guides for the optimum management of recreational lakes can be developed. Eventually, the existing experimental project should be operated on a maintenance basis after most research information has been obtained. Periodically, intensive measurements should then be carried out to determine the long-term behavior of the system.

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SECTION III

INTRODUCTION

The Salt River Valley with Phoenix, Tempe, Mesa, Scottsdale, and Glendale as major cities and a total population of close to one million, is a water-deficient region. In the area served by the Salt River Project alone, about one-third of the roughly one million acre-feet of water annually used by agriculture and municipalities is pumped from the ground water. This is essentially a non-renewable water resource and, consequently, ground water levels in some parts of the valley are declining at about 10 ft/yr.

With the rapid urbanization of the valley, sewage is becoming increasingly significant as a potential water resource. Most of the sewage is treated at the 23rd and 91st Avenue Treatment Plants in Phoenix, which presently handle about 20 and 60 mgd, respectively, or a total of almost 100,000 acre-feet/yr. This volume is expected to increase to about 300,000 acre-feet by the year 2000. Both plants use the activated-sludge process for secondary treatment.

Potential uses of the sewage effluent would be for irrigated agriculture, recreational lakes, and certain industries. Since the average water use by crops is about 4.5 ft/yr, the 300,000 acre-feet of effluent expected by the year 2000 could irrigate about 70,000 acres. Because of the varied agriculture and dense population in the valley, and because canal water is commonly used for irrigation of parks, playgrounds, and private yards, large-scale return of the effluent to the canal system requires tertiary treatment so that it will be aesthetically acceptable and suitable for unrestricted irrigation and primary-contact recreation. The Salt River bed, which is normally dry and traverses the entire valley from east to west at widths of about one-fourth to one-half mile, offers an excellent opportunity to achieve this tertiary treatment by ground water recharge with infiltration basins.

The technical and economical feasibility of tertiary treatment by soil filtration in the Salt River bed, as well as the design and management criteria for an operational system, could only be evaluated with a pilot project. Plans for such a project were formulated and refined in the period 1964-1966. After receipt of a demonstration grant in December 1966 from the then Federal Water Pollution Control Administration, construction of the pilot project, known as the "Flushing Meadows Project," began in February 1967 and was completed in August 1967.

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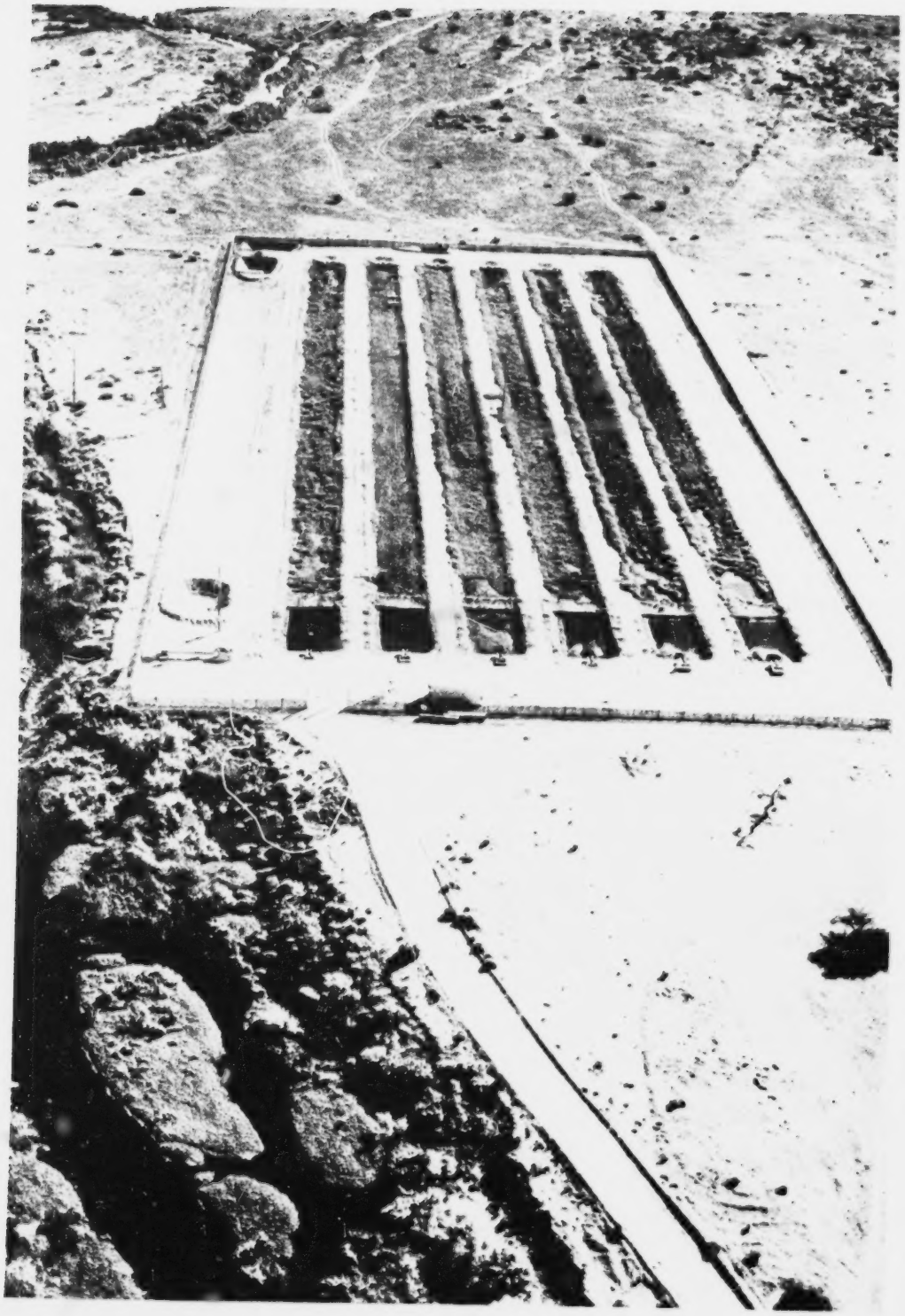

AERIAL VIEW OF FLUSHING MEADOWS PROJECT

SECTION IV

DESCRIPTION OF PROJECT

The Flushing Meadows Project is located in the Salt River bed about 1 1/2 miles west of the 91st Avenue sewage treatment plant. This is an activated sludge plant which discharges unchlorinated secondary effluent into a channel on the north side of the (dry) river bed. The effluent flows westward to Buckeye where some of it is used by the Buckeye Irrigation District.

At the Flushing Meadows Project, effluent is pumped from the channel into a constant-head pipe, from where it flows through an underground concrete pipe to concrete boxes located at the head of each experimental basin (Figure 1). These head boxes are placed above the concrete pipeline and connected thereto with a concrete riser pipe (Figure 2). The flow from the riser into the box is controlled with a valve commonly used in irrigation, a so-called "alfalfa" valve. The water leaves the head box through a triangular, critical-depth flume (10) for measuring and recording the inflow into each basin (Figure 2).

The basins are 20 x 700 ft each with 1:1 side slopes and horizontal bottoms. They are approximately 3 ft deep and spaced 20 ft apart (Figure 1). In February 1968, gravel dams 2 ft high were placed across each basin about 50 ft from the inflow end to provide a presedimentation reservoir at the head of each basin (Figure 2). This was necessary to protect the infiltration basins against high suspended solids loads due to poor effluent quality.

An overflow structure was installed at the downstream end of each basin to control the water depth in the basins. Boards could be inserted or removed to increase or decrease the water depth, respectively (Figure 2). After spilling over the boards, the water passed through a critical depth flume for measuring and recording the outflow. These flumes were of the same type as the flumes at the inflow end of the basins. Thus, constant water depth could be maintained in each basin and the infiltration rate for each basin was calculated as the difference between the inflow and the outflow. Inflow rates were commonly set at 0.4 cfs, the outflow was usually in the 0 to 0.2 cfs range, and the water depths in the basins were maintained at 0.5 or 1 ft. The flow from the outflow end was collected by a concrete pipeline and returned to the effluent channel.

The soil below the infiltration basins consisted of a layer of fine, loamy sand with an average thickness of 3 ft (range 1.3 to 5 ft), underlain by coarse sand and gravel layers to a depth of about 247 ft, where a clay deposit began. This clay deposit is essentially impermeable and it forms the lower boundary of the system.

Mechanical analysis of the loamy-sand top layer indicated that 10% of the soil particles are less than 0.06 mm in diameter and 50% of the

Table 1. Driller's log for East Well and West Center Well.

East Well		West Center Well	
ft		ft	
0-3	fine loamy sand	0-3	fine loamy sand
3-27	sand, gravel and boulders	3-33	sand and gravel
27-30	clean sand, gravel, and boulders	33-44	boulders and gravel
30-49	clean, fine sand with occasional cobbles	44-50	sand and gravel
49-81	clean, fine sand with occasional thin gravel strata	50-57	sand and traces of clay
81-123	clean, fine sand	57-63	coarse, clean gravel
123-126	fine sand with trace of clay	63-72	sand, gravel, traces of clay
126-136	clean, fine sand	72-86	coarse gravel and boulders
136-146	clean sand and gravel	86-98	sand, gravel, and traces of clay
146-197	clean, fine sand	98-100	fine sand
197-200	fine sand and gravel		
200-247	fine sand		
247	start of clay layer		

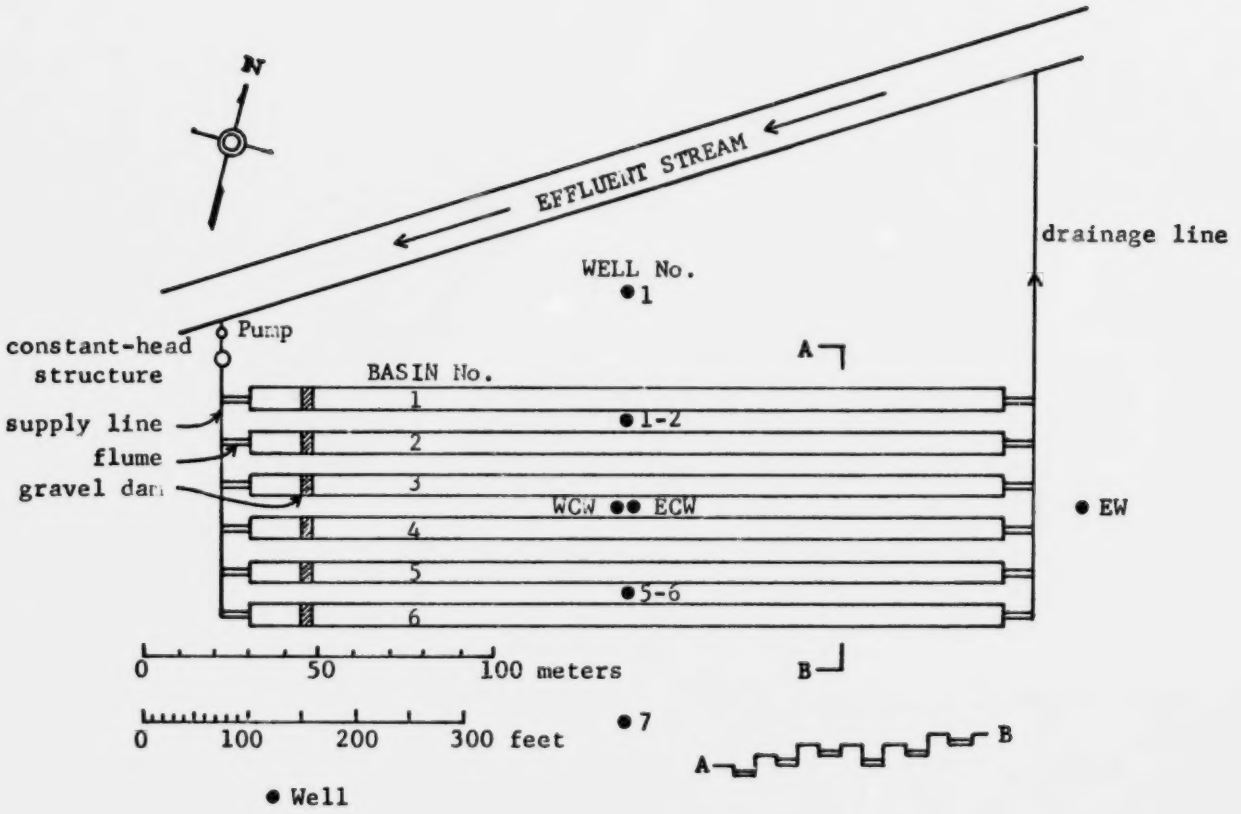

Figure 1. Plan of Flushing Meadows Project.

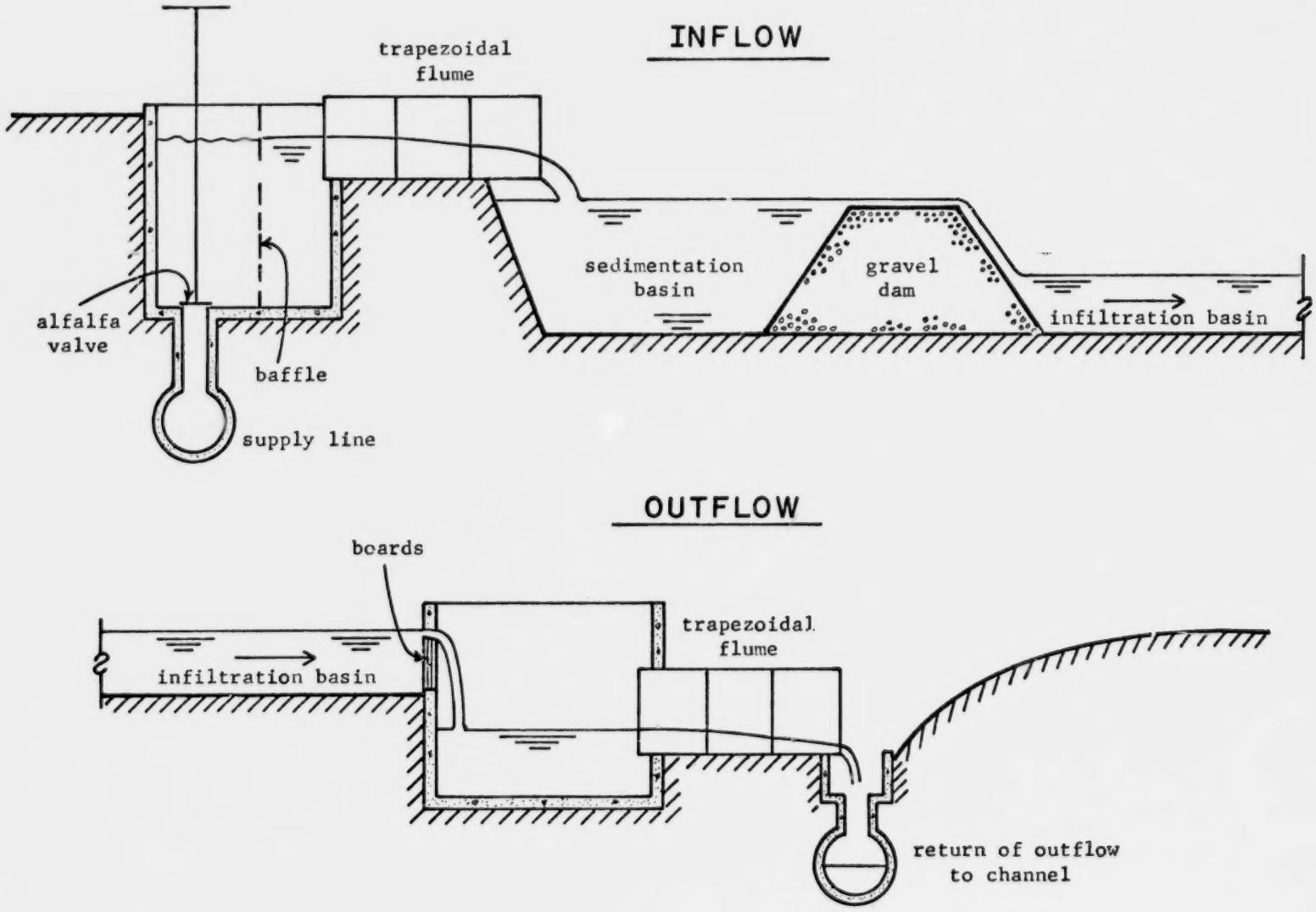

Figure 2. Schematic of inflow and outflow structures for infiltration basins.

particles are less than 0.18 mm in size. The clay content is small and about 2 or 3%. The hydraulic conductivity, K, of the loamy sand at or near saturation is 3 to 5 ft/day and the air-entry value is about -30 cm water, as measured with the double-tube and air-entry permeameter techniques (1, 2, 4, 5).

The sand and gravel layers beneath the loamy-sand top layer are described in Table 1, which is taken from the driller's log for the 100-ft-deep West Center Well and the 247-ft-deep East Well (Figure 1).

The static ground water table was at a depth of about 14 ft below the bottom of the basin at the start of the project in 1967. In the first year of operation, the static water table rose about 4 ft and remained at about 10-ft depth thereafter. The rise of the water table may be attributed to the ground water recharge from the Flushing Meadows Project and to seepage from the effluent channel, which was newly constructed in 1966. Also, the main channel of the Salt River, which is a few hundred yards south of the Flushing Meadows Project, carried water several times in the spring of 1968, which could have recharged the ground water. Observation wells consisting of 6-inch-diameter steel pipe in wells drilled with the cable-tool technique, were installed in a line normal to the basins in the center of the project (Figure 1). Another well, the East Well, is located on the east side of the basins. The East Center Well (ECW), West Center Well (WCW), and East Well (EW), were installed in 1967, the other wells in 1968. The depth of the wells is as follows:

Well	Depth in ft
1	20
1-2	20
ECW	30
WCW	100
5-6	20
7	20
8	20
EW	247

All wells have solid casings open at the bottom with the exception of EW, which was plugged at the bottom and perforated from 10 to 30 ft to yield renovated sewage water for some experimental fish ponds that were installed in 1970.

SECTION V

INFILTRATION STUDIES

1. Basin Management and Infiltration Rates

The effect of basin management on infiltration rate was studied by using different inundation and dry periods, different surface conditions of the soil (bare, gravel-covered, and different types of vegetation), and different water depths. Harrowing was sometimes included as a variable, but usually all basins were cleaned and harrowed at the same time. This was normally done in the spring.

An important factor in the infiltration behavior was the suspended solids content of the secondary effluent, which was low in the summer (5-20 ppm) but high in the winter period November-May (50-100 ppm). These solids were not completely removed by the presedimentation reservoir and a layer of sludge often accumulated on the basin bottoms during the winter period. Upon drying, the sludge layer shrank and broke up into curled-up flakes of about 1 to 5 inches in diameter. Usually, these flakes were removed in the spring, ("shaving" the bottom with a front-end loader was the most effective way), after which the basins were harrowed.

The infiltration rates for the six basins in 1967 are shown in Figure 3, all for a water depth of about 7 inches. The basins were first flooded on 30 August, but equipment failure forced a halt to the pumping. Thus, 22-27 September was the first "real" inundation period. The secondary effluent during this period contained very few suspended solids, and the infiltration rates remained essentially constant. These infiltration rates could be taken as the potential or bench-mark infiltration rates, affected only by the permeability of the soil and not yet by clogging and microbiological processes. The bench-mark infiltration rates were

Basin	Infiltration rate in ft/day
1	3.2
2	4.0
3	3.3
4	3.6
5	3.9
6	3.1

These rates agreed with the range of 3.6 to 6 ft/day calculated by applying Darcy's equation to the 3-ft thick top layer of the loamy sand using the hydraulic conductivity values determined with the double-tube and air-entry permeameter techniques.

The infiltration rates for the next inundation period, 3-6 October were lower, probably because of a 0.5-inch rain which fell just before the basins were flooded. On 12 October, all basins were harrowed,

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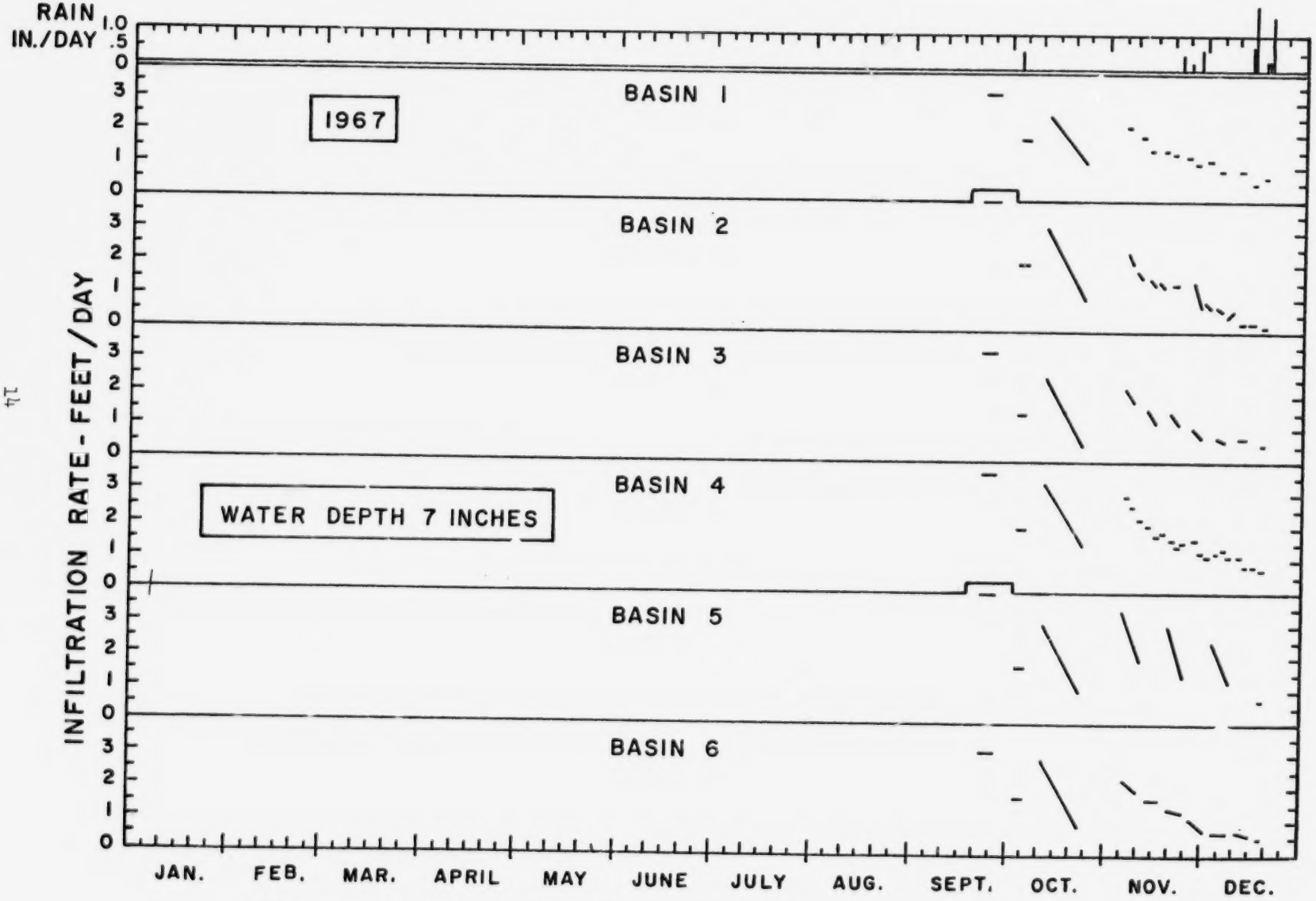

Figure 3. Infiltration rates for recharge basins in 1967.

except basin 5 to see how effective harrowing was in restoring infiltration rates. Expressing the initial infiltration rates for the next flooding period, 12-24 October, as a percentage of the bench-mark rates for the first period 22-27 September, shows that the lowest percentage is obtained for basin 5, which was not harrowed prior to flooding (77% compared to 80-94% for the other basins). Thus, the low infiltration rates for 3-6 October were likely due to surface sealing and crusting caused by the rain, and harrowing was effective in breaking up this crust. The infiltration rates for the period 12-24 October decreased almost linearly with time. This decrease was probably mainly due to the solids content in the effluent, which had started to increase to its winter level.

On 3 November, all basins were harrowed again and different inundation schedules were used for the rest of the year (Figure 3). The suspended solids content of the effluent was high and had a significant effect on the infiltration rates. The infiltration rates again decreased almost linearly with time during inundation and dry-up periods of at least 9 days were required to obtain reasonable infiltration recovery (Figure 3).

Infiltration rates for 1968 are shown in Figure 4. On 6 and 7 February, the basins were raked to remove the sludge flakes. On the same day, gravel dams were placed about 50 ft from the inlet end to create presedimentation basins (Figure 2). These basins were effective in reducing the solids load on the infiltration basins. The sludge removal and drying gave excellent infiltration recovery, as shown by the high infiltration rate for the period 7-10 February. On 20 February, all basins were harrowed with a tooth harrow. The low infiltration rate for basin 1 for the next infiltration period cannot be explained and may be erroneous.

On 16 April, the water depth in the basins was increased from the 7 inches used so far to 13 inches by adding a board in the overflow structure (Figure 2). This almost doubled the infiltration rate (Figure 4), indicating that the infiltration reduction in the basins was mainly caused by clogging of the surface layer of the soil.

The basins were swept on 7 May with a power lawn sweeper to remove the sludge flakes. Next, all basins were harrowed with a tooth harrow. Basins 3, 4, 5 and 6 were seeded with a mixture of giant and common bermudagrass and then irrigated with about 6 inches of effluent every 2 or 3 days. These irrigations, for which the infiltration rates were estimated at 2 ft/day, are shown as dots in Figure 4. Test strips of other vegetation, including sudangrass, tifway, fescue, and blue panicum, were also planted.

Basin 2 was covered with a 2-inch layer of coarse sand ("concrete" sand) and topped with a 4-inch layer of 3/8-inch gravel on 23 May. Basin 1 was left in bare soil condition. All basins received 1 ft of water on

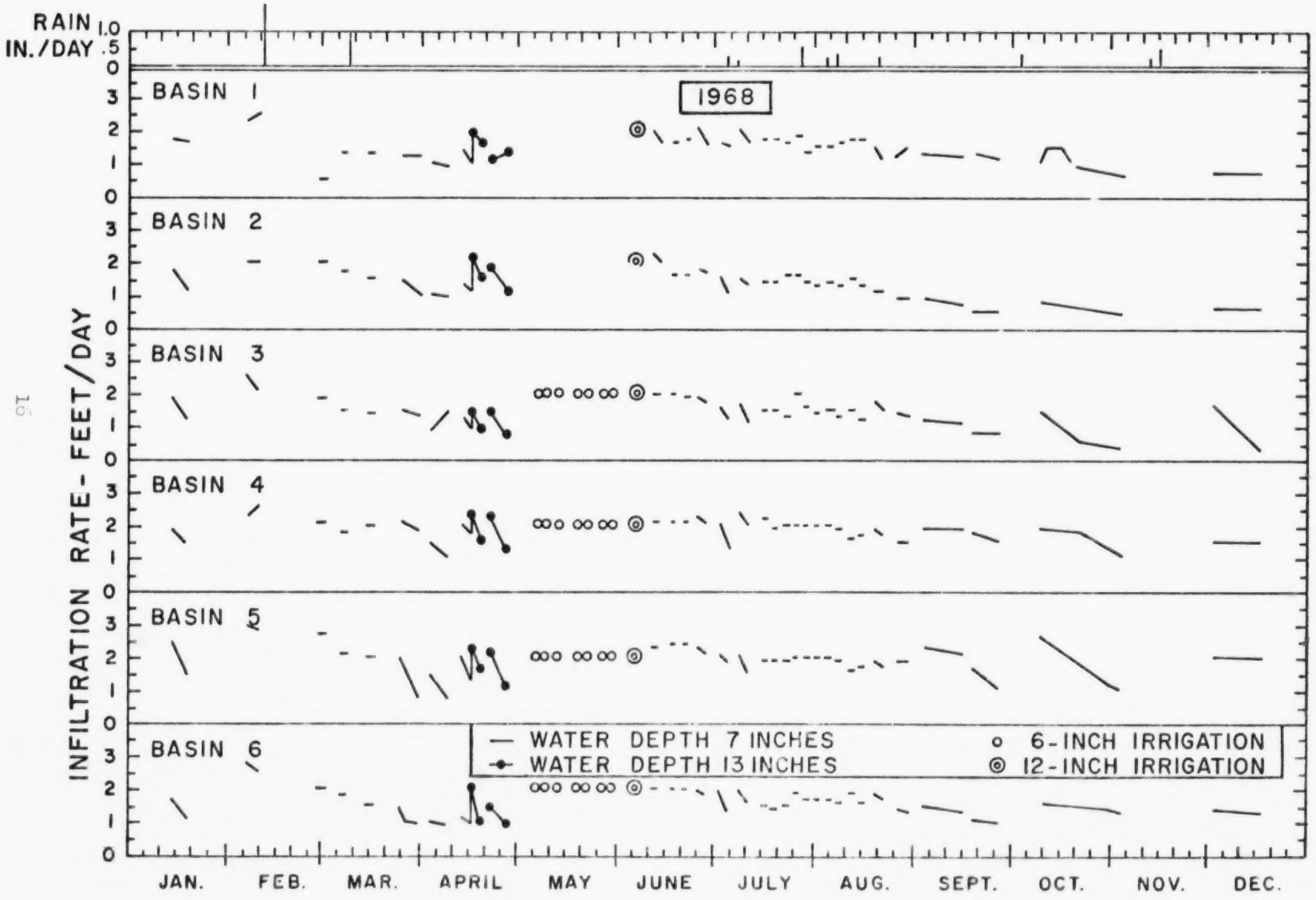

Figure 4. Infiltration rates for recharge basins in 1968.

7 June and a sequence of short inundation periods was started for all basins on 12 June, using a water depth of 7 inches.

The grasses in basins 3, 4, 5, and 6 reached a mature stand in August with the giant bermudagrass emerging as the dominant species. To evaluate the effect of the grass cover on the quality of the effluent as it flowed through the grass, "series" flow was started to increase the distance of overland flow for the effluent. For this purpose, the basins were split into two groups of three with the flow going in serpentine fashion through the three basins in each group, as shown in Figure 5. Flumes were installed where the effluent flowed from one basin to the next, so that inflow and outflow were measured for each basin. The water depths in the basins during this period of series flow were as follows:

Basin	Water Depth in Ft
1	0.7
2	0.7
3	0.85
4	0.85
5	0.85
6	0.95

The water depth in the gravel basin (basin 2) refers to the original soil surface. The grass in the vegetated basins was sufficiently tall to be well above the water level.

Rather long inundation periods were selected for the series-flow sequence because the effect of flow through the grass on the quality of the effluent could be expected to increase with time, as biologically active films developed on the grass stems. For the period 15 August to 17 December, the secondary effluent was of good quality and had a low suspended solids content. Nevertheless, the grass basins 3 and 6, which received effluent directly from the channel, had fairly low infiltration rates (Figure 4).

The effect of surface condition on infiltration rate was evaluated for basins 1, 2, 4, and 5, which received effluent that had traveled through at least one basin of grass and therefore had lost most of its suspended solids. Table 2 shows the accumulated infiltrations for the period 20 August-17 September. In the second column, these values are adjusted to a common water depth of 0.7 ft in the basins (assuming a linear relation between infiltration rate and water depth). In the third column the bench-mark infiltration rates for the period 22-27 September 1967 are listed. The fourth column shows the ratio between adjusted and bench-mark infiltration rates and the fifth column shows these ratios with the highest ratio arbitrarily set at 100.

The highest relative infiltration index is obtained for the grass basins with values of 100 and 94 (Table 2). Next comes the bare soil

Table 2. Effect of surface condition on infiltration rate (1968).

Basin	Accumulated infiltration for 20 August-17 December feet	Adjusted accumulated infiltration feet	Bench-mark infiltration rates feet/day	Ratio of adjusted to bench-mark infiltration	Index of relative infiltration
1 (bare soil)	72	72	3.2	22.5	84
2 (gravel cover)	45	45	4.0	11.2	42
4 (grass)	111	91	3.6	25.3	94
5 (grass)	127	105	3.9	26.9	100

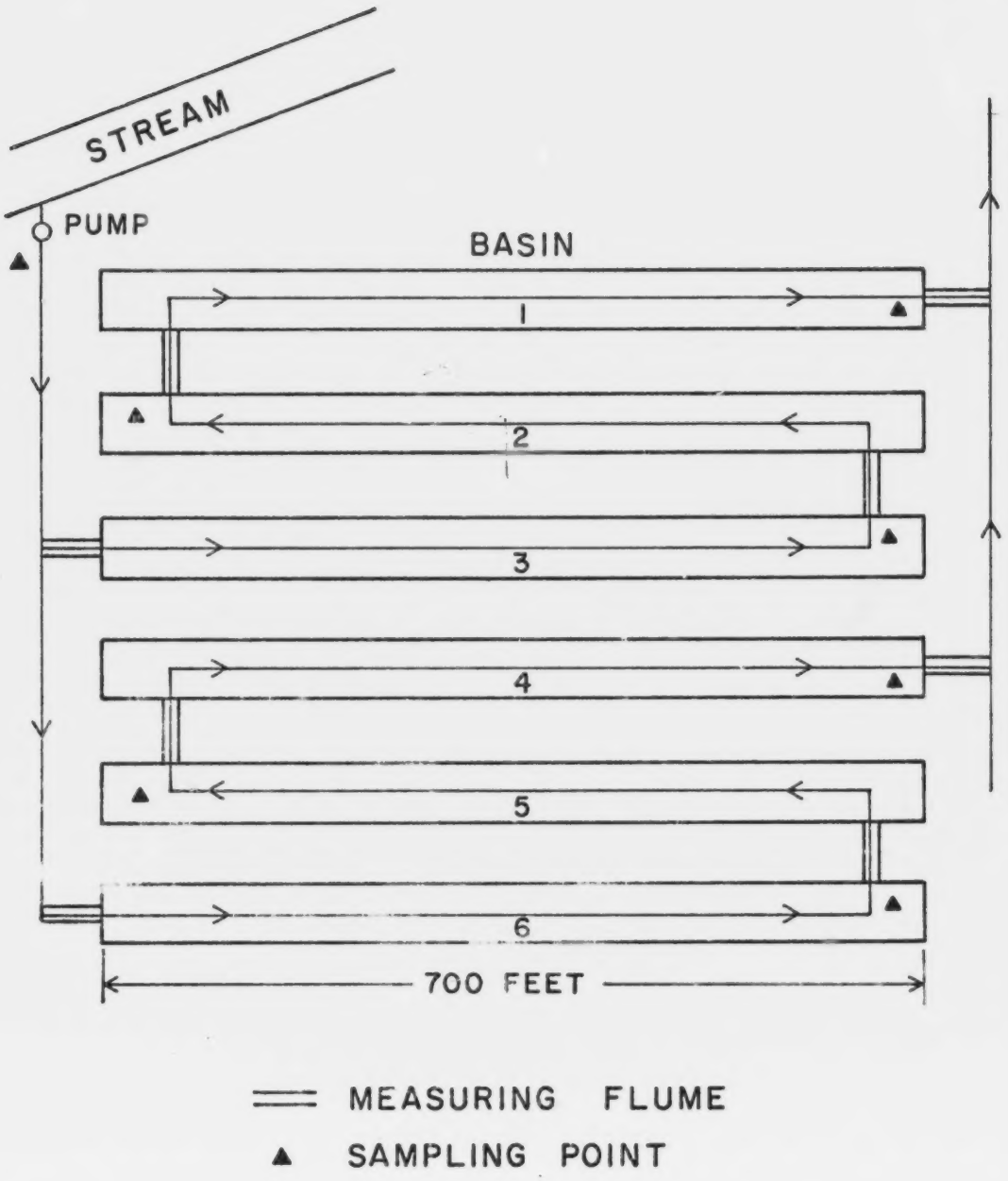

Figure 5. Diagram of series flow with measuring and sampling points.

basins with 84. The gravel basin is lowest with 42. The higher infiltration rates in the grass basins are probably due to reduced clogging of the soil surface by suspended solids and algae. The negative effect of the gravel layer on infiltration is probably due to a "mulching" action of the gravel layer and resulting poor drying of the underlying soil. Also, dust and other solids may have entered the gravel layer and settled on the gravel-soil interface, creating a clogged layer which could not be dried or broken up mechanically.

The giant bermudagrass appeared to be the most suitable grass. It dominated the common bermuda and developed a vigorous stand that was sufficiently dense for filtration and shading, yet sufficiently open for the effluent to flow through, rather than over, the grass. This made it possible to use inundation periods as long as a month without adverse effects on the condition of the grass. Excellent growth was also exhibited by tifway. However, the sod was too dense to permit much flow of the effluent through the grass. The resulting increase in water depth and complete inundation then caused the tifway to die, particularly at the inflow end where solids settled on the grass. The sudangrass also grew well, but the stand was not sufficiently dense to provide good filtration and shading. Also, it died after reaching maturity. The other grasses tested (blue panicum and fescue) failed to survive. None of the grasses were mowed in 1968.

At the beginning of 1969, basin 1 was still in bare soil condition, basin 2 had a gravel layer, and basins 3, 4, 5, and 6 had the dead bermudagrass left from the 1968 growing season. The bermuda straw lay flat and became covered with sludge due to the poor effluent quality in the first half of 1969. Because of this, the infiltration rates were relatively low, even when the water depth was 13 inches (Figure 6). Just before the flooding period starting 3 March, basins 3, 4, 5, and 6 were burned. Only the top of the bermuda straw, which formed a mat about 4 inches thick, was dry enough to burn effectively.

In the first week of April, basin 1 was swept with a power lawn sweeper to remove most of the dry sludge flakes, and then harrowed with a spike tooth harrow. Basin 2 was left unchanged and basins 3, 4, 5, and 6 were burned several times and harrowed between burnings to speed up drying of bermuda straw. This procedure was fairly effective and when the next flooding period started on 7 April, the previously vegetated basins were essentially in bare soil condition.

On 14 April, basin 6 was harrowed and seeded with rice (variety Caloro). All basins were then inundated for 2 days. On 21 April, basins 1, 3, 4, and 5 were harrowed again and basin 5 was seeded with rice. For the next few months, short, frequent inundation periods were used for basins 5 and 6 to promote germination and early growth of the rice. In July, the rice was sufficiently tall to permit longer inundation periods for the rest of the growing period.

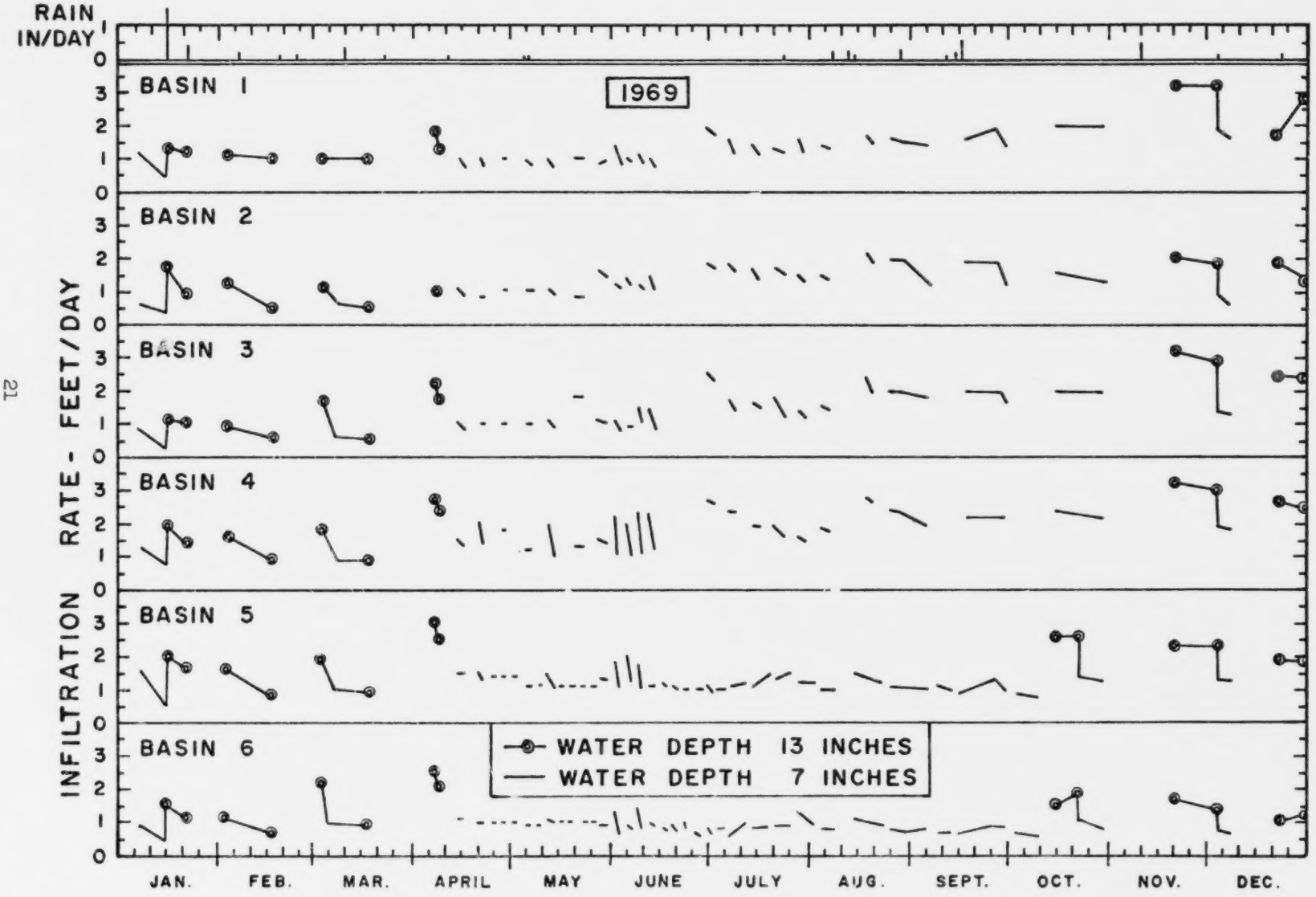

Figure 6. Infiltration rates for recharge basins in 1969.

Because of the continuous high suspended solids content of the secondary effluent, short inundation periods were also maintained in basins 1, 2, 3, and 4 for the April-July period. At the beginning of a flooding period, the infiltration rates were relatively high, but they declined rapidly because of the buildup of a sludge layer on the bottom of the basins. Drying was effective in restoring infiltration rates.

The giant bermudagrass in basins 3 and 4 made a slow comeback. The tifway had completely died and there were only sporadic tufts of giant bermudagrass. During the frequent short inundations in May and June, however, the bermudagrass spread rapidly, especially from the banks. Also, native grasses, mainly Mexican sprangletop (Leptochloa uninervia) and barnyard grass (Echenochloa crusgalli) came in and exhibited a luxurious growth, particularly in basin 1, which previously had been kept in bare soil condition. A third grass, probably blue panicum, also volunteered in basins 1, 3, and 4. The condition of the basins in summer and fall of 1969 was as follows:

Basin 1, dense stand of sprangletop, barnyard grass, and blue panicum

Basin 2, gravel layer

Basins 3 and 4, giant bermudagrass with sprangletop and barnyard grass and some blue panicum

Basins 5 and 6, rice with giant bermuda and some sprangletop, barnyard grass, and blue panicum.

A 2-week dry-up period was held in the second half of June for basins 1, 2, 3, and 4. This was effective in restoring the infiltration rates which had become low because of the sludge accumulation in the basins. The effluent quality improved at this time, so that normal infiltration rates were again maintained.

On 9 August, a pump failure occurred so that the rice was dry for almost 6 days. This happened during a very hot period and it caused visible damage to the rice. Although the rice seemed to recover fully, not much grain was formed and also because of lodging later on, no attempt was made to harvest the grain.

A sequence of long inundation and dry-up periods was started in August, with the rice basins receiving shorter dry ups than the other basins until 15 October. On 20 November, the vegetation in basins 1, 3, and 6 was mowed, baled, and removed. Basins 4 and 5 were not mowed. Thus, the condition of the basins at the start of the winter was as follows:

Basin 1, stubble of sprangletop and barnyard grass

Basin 2, gravel layer

Basin 3, stubble of bermudagrass

Basin 4, bermudagrass straw (about a 4-inch thick layer)

Basin 5, rice straw (about a 2- to 3-inch layer)

Basin 6, rice stubble.

For the next flooding period, starting 21 November, a water depth of 13 inches was employed. On 4 December, the water depth was reduced to 7 inches. After the decrease in the water depth, the infiltration rates were on the average 57% of the values prior to reducing the water depth. Theoretically, a reduction of close to 50% can be expected if surface clogging is the restricting factor. If the fine-sandy-loam top layer had become less permeable over its entire thickness, Darcy's equation shows that the infiltration rates at a water depth of 7 inches would be 80% of the rates at a water depth of 13 inches. Since the actual reduction is closer to 0.5, most of the infiltration decrease during flooding is thus due to clogging of the surface layer of the soil. Applying a similar analysis to the basin covered with a gravel layer shows that the reduction of the infiltration in the gravel-covered basin is due to clogging of the surface of the original soil, just below the soil-gravel interface.

Rainfall, which adversely affected the infiltration rate during flooding when it occurred during the preceding dry-up period and the basins were in bare soil condition, did not affect the infiltration rate when the basins were vegetated. For example, on 15 September, a 0.6-inch rain fell just before the start of a new inundation period. As shown in Figure 6, this rain had little or no effect on the infiltration rate. Thus, the reduction in infiltration rate due to rain must be attributed to the mechanical impact of the raindrops on the soil surface and the resulting translocation of fine particles and sealing. In areas with high rainfall or where the sewage effluent or other wastewater is applied with sprinklers through the land, a vegetated surface may be required to avoid direct impact of the drops on the soil surface.

The accumulated infiltration for 1969 is shown in Figure 7. Dividing the annual infiltration by the bench-mark infiltration for each basin and setting the ratio for the basin with the highest relative infiltration rate at 100, according to the procedure used in Table 2, yields the following indexes of infiltration for 1969:

Basin	Index of Relative Infiltration
1	95
2	65
3	92
4	100
5	92
6	85

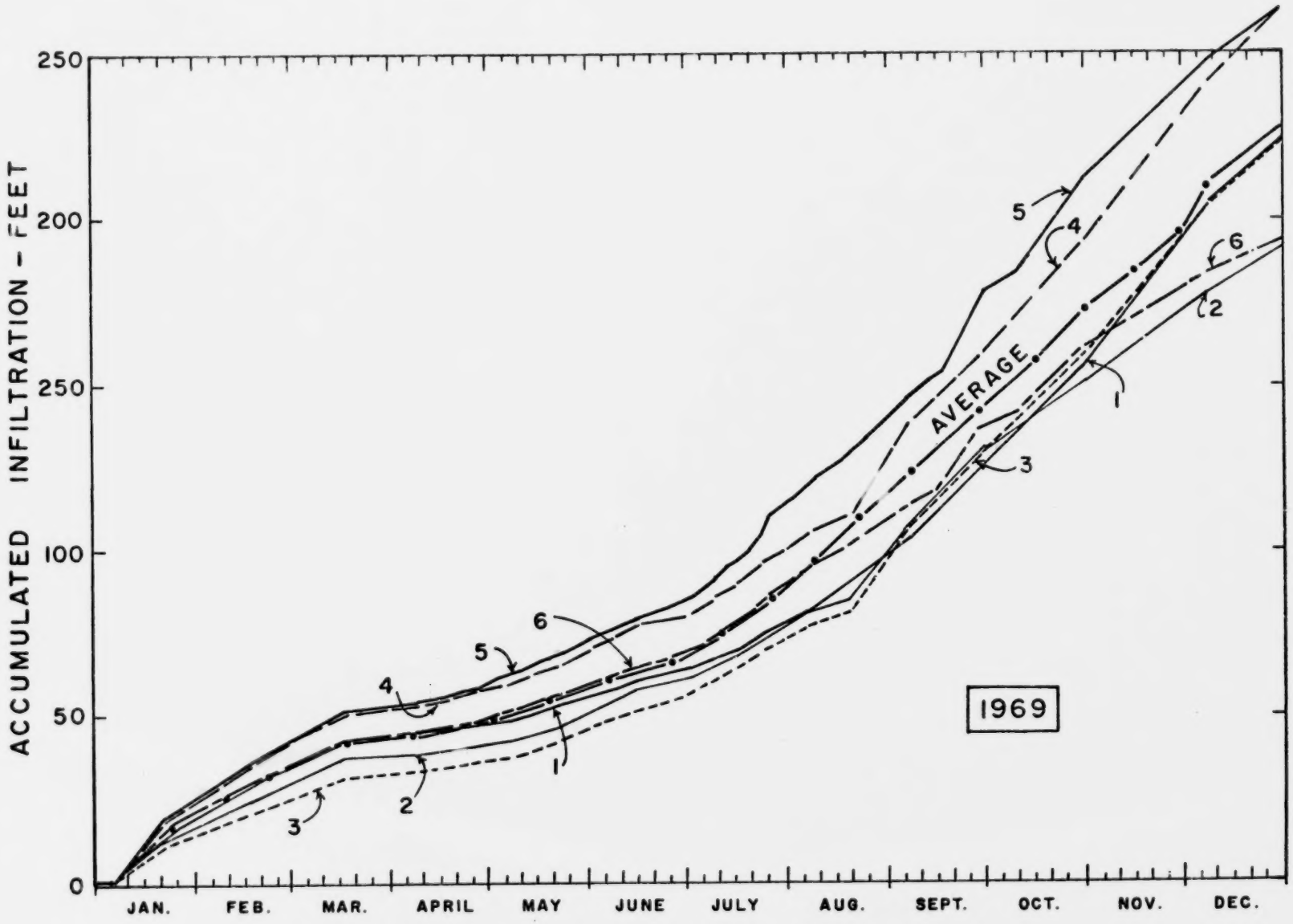

Figure 7. Accumulated infiltration for recharge basins in 1969.

The vegetated basins all appear to have relative infiltration indexes between 85 and 100. The rice basins are in the lower part of the range, probably because of the short dry-up periods used in the summer and fall. The gravel basin again had the lowest relative infiltration with an index of 65. The index for basin 1 was 95, which is higher than the index of 84 obtained in 1968 when no vegetation was allowed to develop in this basin.

2. Pressure-Head and Water-Content Measurements

To evaluate flow regimes in the loamy-sand top layer, pressure heads were measured at different depths with tensiometers during inundation and dry-up periods. Water contents of the loamy sand were measured with the neutron method. The loamy-sand layer is the top layer of the soil profile beneath the basins and its average thickness is 3 ft.

Measurements taken in November and December 1967 indicate that the pressure heads in the loamy-sand top layer were always negative, except for the first few inches or so shortly after the start of a new inundation period. The pressure heads decreased (became more negative) as inundation progressed. Sometimes the pressure heads decreased rapidly during flooding (Figure 8) indicating severe clogging of the soil surface above the tensiometer profile. Sometimes the decrease was more gradual (Figure 9). For both cases, the pressure heads also decreased from one inundation to the next, indicating increased clogging at soil surface.

The pressure-head measurements at different depths enabled the calculation of the unsaturated hydraulic conductivity, K_u, during inundation for each depth increment between tensiometers. This was done for basin 2 in the fall of 1967 when the basin was not yet covered by a gravel layer. The infiltration rates in these calculations were taken as the average infiltration rates for the entire basin as calculated from the inflow-outflow records. The results (Figure 10) show that K_u of the 8- to 40-inch layer remained essentially equal to the infiltration rate. Thus, the hydraulic gradient in this layer was essentially one and the increasing desaturation of this layer was due to a decrease in infiltration rate caused by clogging of the soil above the 8-inch-depth level. The value of K_u of the 4- to 8-inch layer was less than the infiltration rate, and K_u of the 0- to 4-inch layer was even lower. Thus, the top layer had the lowest hydraulic conductivity and most of the K-reduction was concentrated in the surface layer of the soil. This is also indicated by the essentially linear relationship between infiltration rate and water depth in the basins previously discussed.

When a new flooding period is started, "final" infiltration rate and essentially unit gradient in the loamy-sand top layer should be reached in less than a day. If soil clogging has not yet begun, the "final" infiltration rate can thus be used as an estimate of K of the loamy sand at saturated or near-saturated conditions. The bench-mark infiltration rates can be used for such a purpose, which would yield a

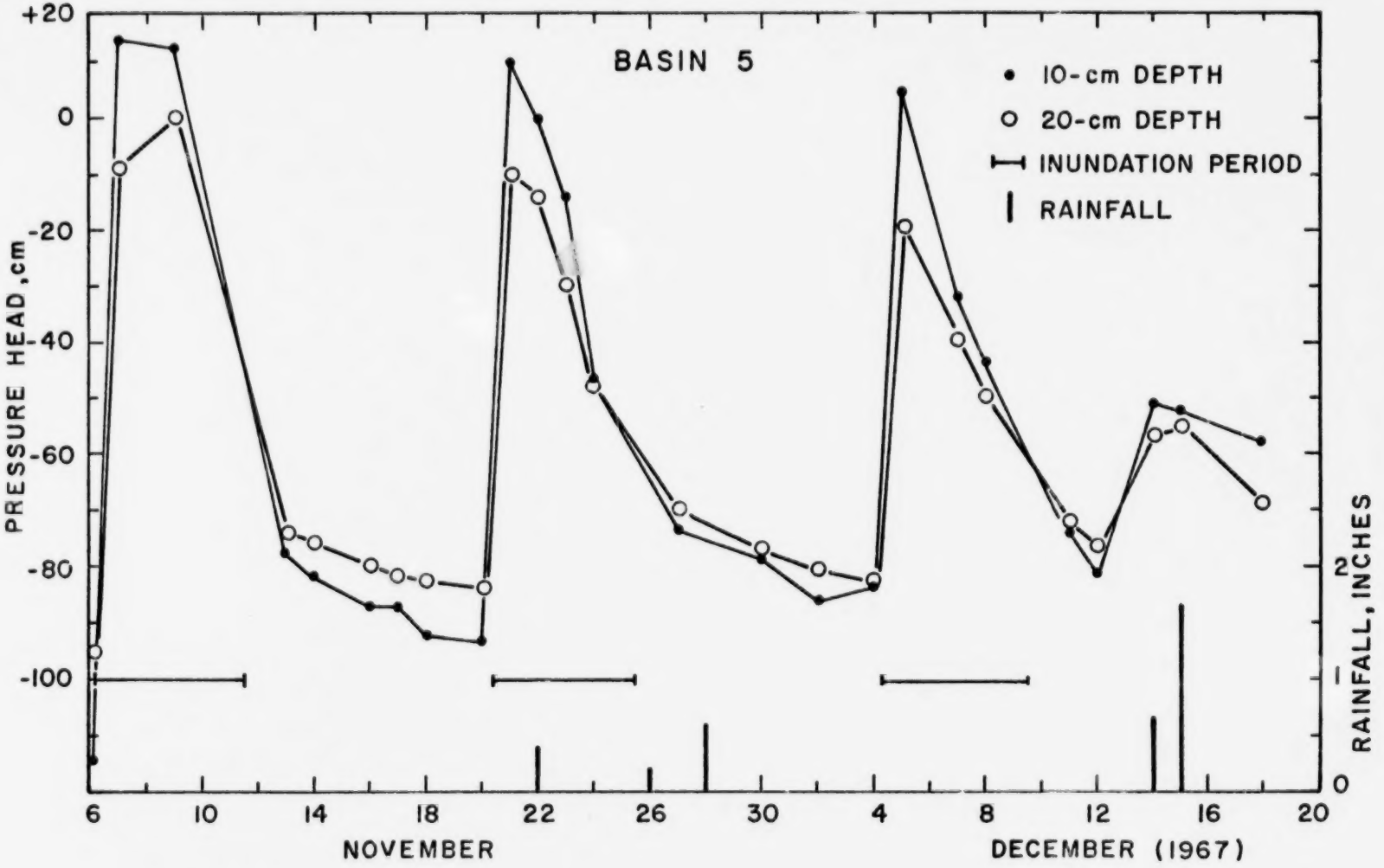

Figure 8. Pressure heads of water in soil beneath basin 5 during flooding and dry up.

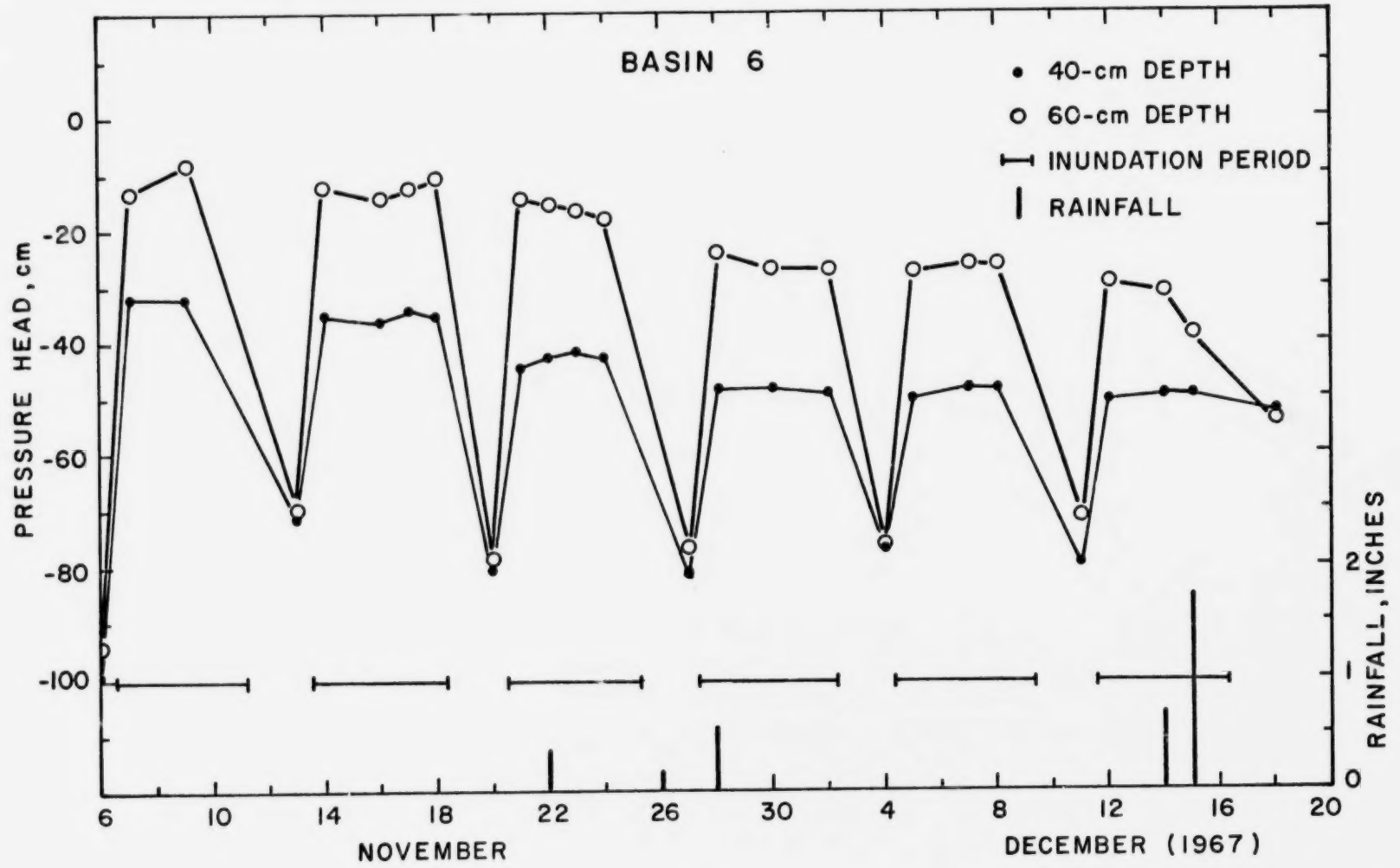

Figure 9. Pressure heads of water in soil beneath basin 6 during flooding and dry up.

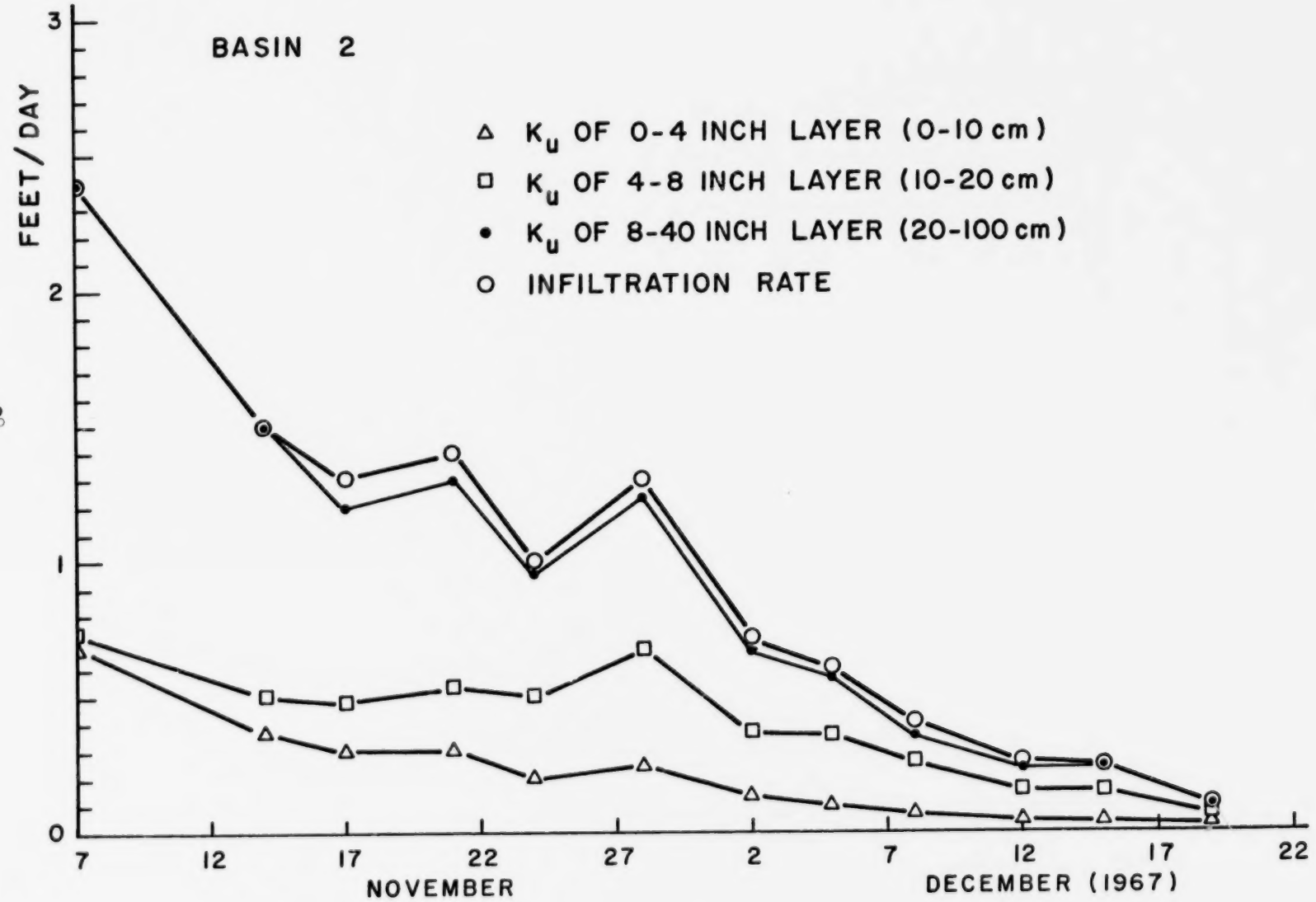

Figure 10. Infiltration rate and K_u at different depths below basin 2.

K-value of 4 ft/day for the loamy sand in basin 2. The unsaturated hydraulic conductivity, K_u, of the loamy sand can be taken from Figure 10 and related, for example, to the tensiometer readings used to calculate K_u. The resulting relation between K_u and (negative) pressure head is shown in Figure 11.

Combining the tensiometer readings with the water-content measurements yields the relation between water content and pressure head of the loamy sand. This relationship is shown in Figure 12 for three depths in basin 5.

The curves in Figures 11 and 12 describe the unsaturated hydraulic conductivity and water content characteristics of the loamy-sand top layer. This information would enable the theoretical analysis of the flow system in the top layer under intermittent inundation. Such analyses may yield predictions of water and air movement into soil and they may be used, for example, to develop guides for inundation and dry-up schedules whereby oxygen demand of the system is matched to oxygen supply under a variety of conditions.

3. Basin Management for Maximum Hydraulic Loading

The general pattern of the infiltration behavior of the recharge basins was an essentially linear reduction in infiltration rate from 2-3 ft/day at the beginning of an inundation period to 1-2 ft/day after approximately 3 weeks inundation, using a water depth of about 1 ft. Dry-up periods of less than a week were generally not effective in restoring infiltration rates. Most of the infiltration recovery seemed to take place in the second week of the dry up. Thus, the infiltration recovery curve is S-shaped, which was confirmed with infiltrometer measurements. The linear reduction in infiltration rate during flooding and the S-shaped recovery during dry up are schematically shown in Figure 13, where the infiltration rate at the start of an inundation period was arbitrarily set at an index of 100.

Assuming full infiltration recovery after 12 days dry up in the summer and 20 days in the winter, long-term infiltration rates (which include the time that the basins are not inundated) were calculated for different lengths of the inundation period. The resulting curves, shown in Figure 14 with the infiltration rate at the start of the inundation period again taken as 100, have flat peaks with a maximum long-term infiltration rate, or hydraulic loading, at inundation periods of about 24 days in the summer and 30 days in the winter. Because the infiltration recovery may be slower after a long inundation than a short inundation, it would probably be best to use inundation periods that are on the left side of the maximum. Thus, maximum long-term infiltrations will probably be obtained with inundation periods of 16 to 24 days, alternated with dry-up periods of 12 days in the summer and 20 days in the winter.

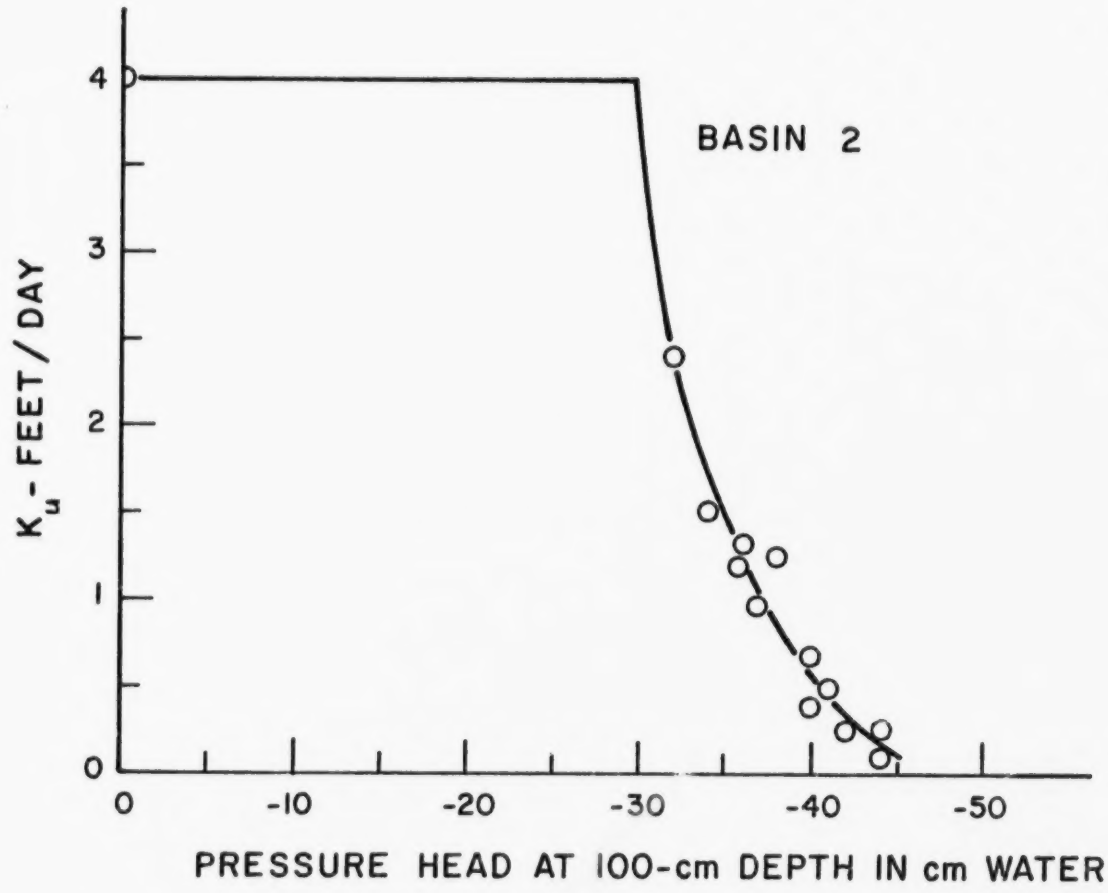

Figure 11. K_u as a function of soil-water pressure-head for soil in basin 2.

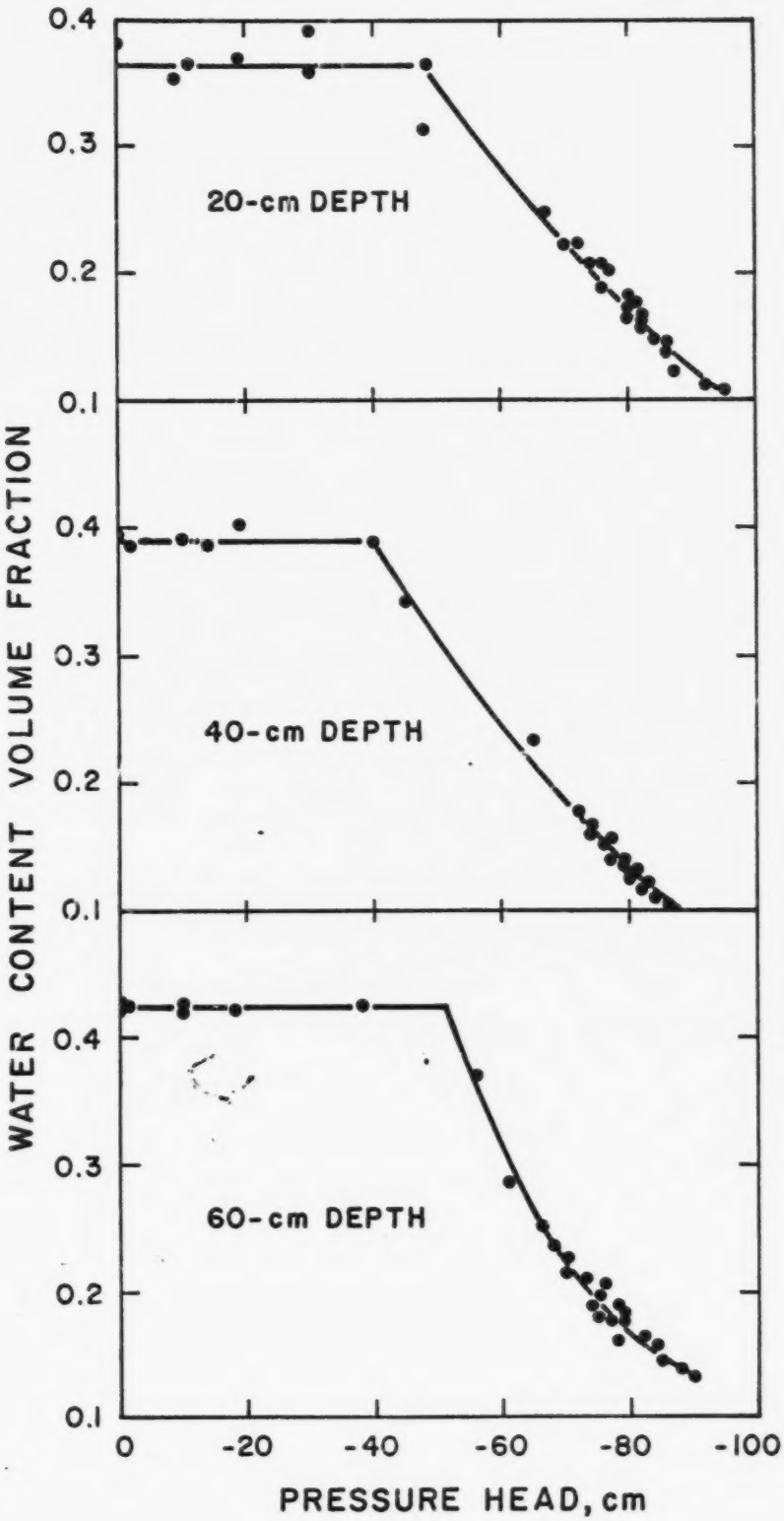

Figure 12. Relation between water content and pressure head for soil in basin 5.

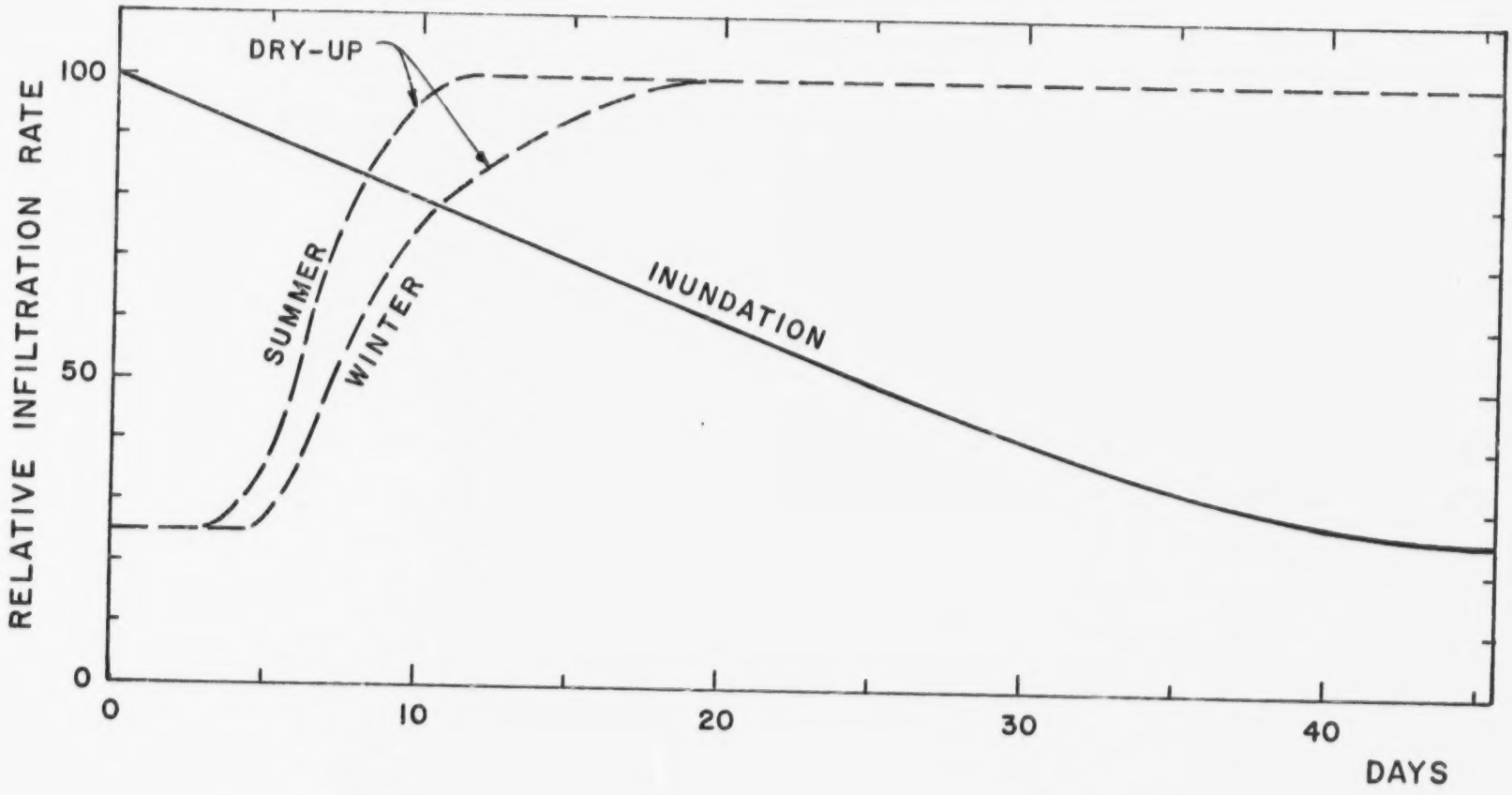

Figure 13. Schematic presentation of infiltration decrease during inundation and recovery during dry up.

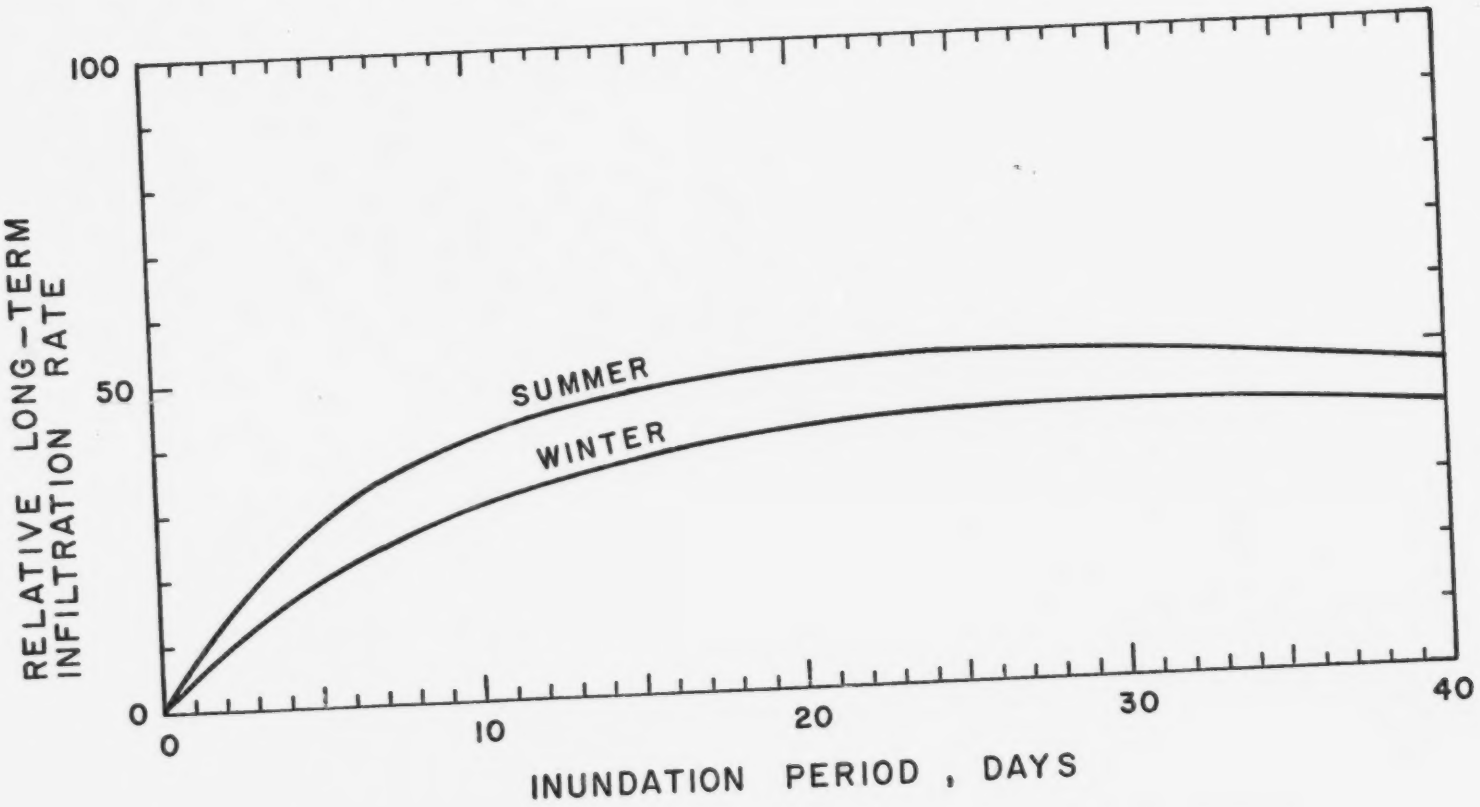

Figure 14. Long-term infiltration rate in relation to length of inundation periods.

The average height of the maxima of the summer and winter curves in Figure 14 is at an infiltration index of about 45. The infiltration rate at the start of a new flooding period is of the order of 2.5 ft/day, using a water depth of 13 inches in the basins. Thus, the average long-term infiltration rate for the inundation schedules mentioned in the previous paragraph will be about 1.12 ft/day. This yields an annual loading rate of 410 ft (410 acre-feet per acre per year). This loading was essentially achieved in 1970, when the average accumulated infiltration for the six basins was 400 ft.

Since complete infiltration recovery to the original values may not always be obtained, it may be necessary to periodically include an extra long dry-up period. The best time for such a dry up would be when drying conditions are favorable, such as in the summer or during periods of low rainfall.

The gravel layer clearly yielded lower infiltration rates in the basins, contrary to what has been observed for other recharge facilities such as the Peoria recharge pit in Illinois and the Whittier Narrows recharge basin near Los Angeles. Covering the basins with a gravel layer is, therefore, not recommended for systems similar to the Flushing Meadows Project.

Vegetation in the basins yielded higher infiltration rates in the summer and fall, when the vegetation had reached a mature stand. However, the water depth in vegetated basins is restricted to avoid complete submergence of the vegetation. Thus, the higher infiltration rate of the vegetated basins could also have been obtained with bare soil basins, simply by increasing the water depth. Also, vegetated basins require a sequence of short, frequent, and shallow inundations in the spring to get the vegetation established, whether from seed or from winter dormancy. The total infiltration during such a sequence will tend to be less than that for a sequence of longer inundation and dry-up periods, as could be used with a bare soil basin. Thus, bare-soil basins may yield equal or even higher annual infiltration amounts than the vegetated basins, because of the greater water depths and longer spring inundation periods that can be employed.

Vegetation may be desirable in areas with high rainfall or where the waste water is supplied with sprinklers, to protect the surface of the soil against the impact of the water drops. Also, vegetation may be preferred because of considerations other than infiltration rates, for example, nitrogen removal from the soil or utilization of the infiltration basins for agricultural production. Thus, the choice between vegetated or nonvegetated basins is governed by a number of factors, which should be considered for each individual system.

Scraping, sweeping, harrowing, or disking the basin bottoms is necessary when sludge accumulations restrict the infiltration rates, and particularly the infiltration recovery during dry ups. "Shaving" the bottom

with a front-end loader or sweeping with a power sweeper effectively removes dried sludge flakes. If the sewage effluent has a low suspended solids content (for example, less than 10 mg/l), sludge removal and harrowing may only be necessary once a year or once every few years. Suspended solids concentrations of 50-100 mg/l in the effluent may require sludge removal every few months.

SECTION VI

AQUIFER STUDIES

1. Hydraulic Conductivity of Aquifer and Flow System

Analog Studies. Hydraulic properties of the aquifer beneath the infiltration basins were evaluated so that the flow system below the water table could be determined and the underground detention times of the renovated water from the various observation wells could be estimated. Also, knowledge of the hydraulic properties of the aquifer was necessary for the design of the large-scale effluent recharge and reclamation system in the Salt River bed envisaged for the future.

Because of the stratified nature of the alluvial deposits (Table 1), the aquifer could be expected to behave as an anisotropic medium with the hydraulic conductivity in horizontal direction, K_h, greater than the hydraulic conductivity in the vertical direction, K_v. Values of K_h and K_v were obtained from the response of the water levels in ECW and WCW to infiltration of effluent in the basins.

The water levels in the observation wells rose with the start of a new inundation period, reached a pseudo-equilibrium level after a few days, and declined slowly as the infiltration rates in the basins decreased due to soil clogging. When the inundation of the basins was stopped, the water levels in the observation wells dropped in typical "decay" fashion to about the same levels as before the inundation period. This behavior is illustrated in Figure 15 for ECW. The drop of the water level in ECW between 3 and 5 December was caused by reduction in infiltration rate due to changing the water depth in the basins from 13 to 7 inches on 4 December. The elevations in Figure 15 are with respect to a local bench-mark. The elevation of the bottom of the recharge basins with respect to the bench-mark is about + 8 ft.

For the inundation period 22-27 September 1967, the infiltration rates were high and remained essentially constant at about 3.5 ft/day (average for the 6 basins). Thus, conditions during this period were favorable for evaluating K_h and K_v from the water level rises at pseudo-equilibrium. These rises were 2.7 ft for ECW, and 0.7 ft for WCW.

To evaluate K_h and K_v of the aquifer from the pseudo-equilibrium water level rises in ECW and WCW, a vertical cross section of the aquifer was simulated on a resistance network analog, assuming a horizontal water table and taking the impermeable boundary at 247-ft depth. The cross section was taken normal to the basins, in the line of observation wells 1 through 8 (Figure 1). The portion of the horizontal water table beneath the recharge basins was treated as a source, and the rest of the water table as a sink. The aquifer was assumed to be of infinite lateral extent, which was represented by a termination zone according to the procedure described in a previous paper (6). Assuming that the aquifer was uniformly anisotropic, different ratios of K_h/K_v were

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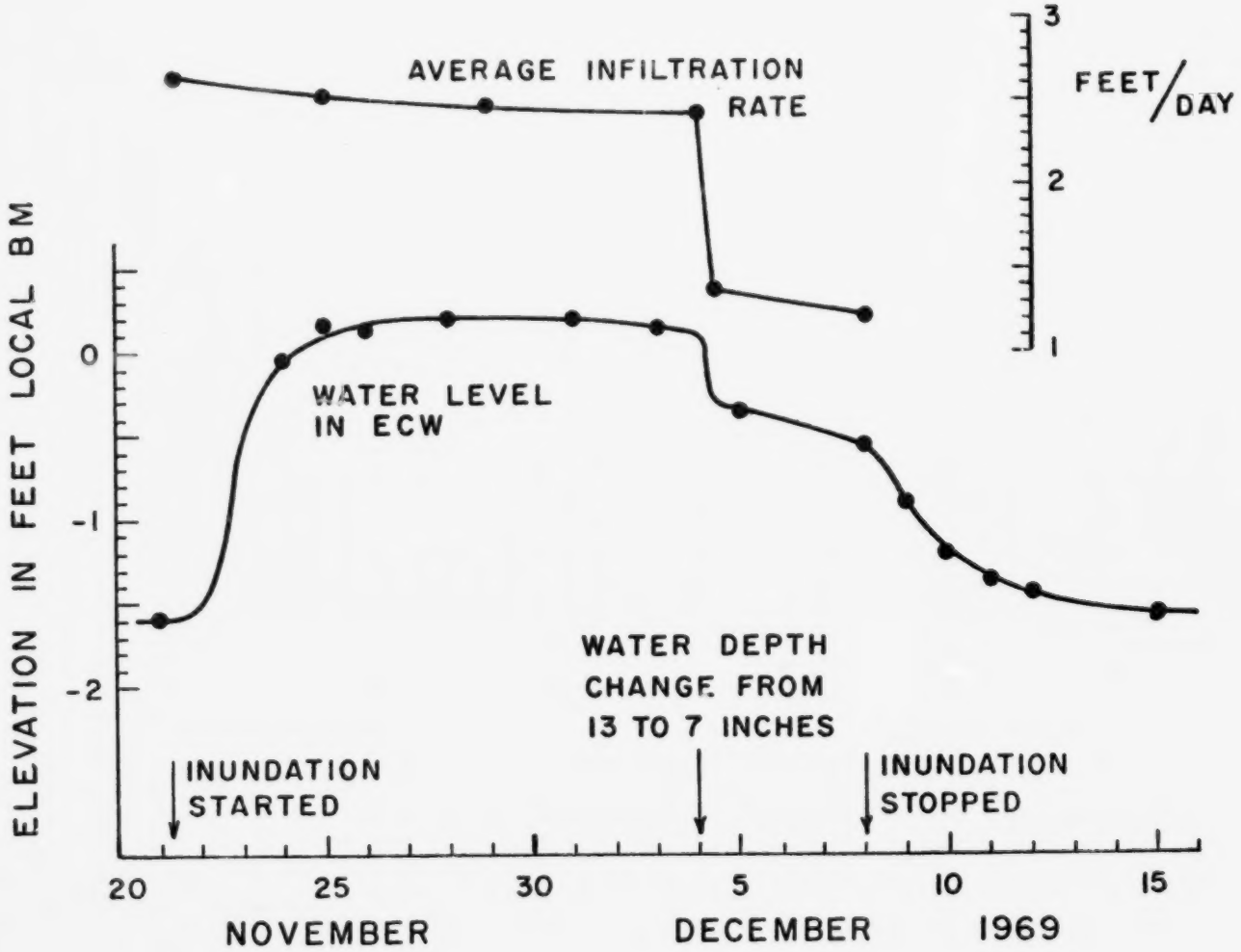

Figure 15. Response of water level in ECW to infiltration.

simulated until the voltage rises at the points representing the 30-ft and 100-ft depths in the center of the system (corresponding to the bottom of ECW and WCW, respectively) agreed with the water level rises of 2.7 ft and 0.7 ft, respectively, measured in the field. This agreement was achieved when K_h/K_v was taken as 16.

With the proper value of the K_h/K_v-ratio thus established, the pseudo-equilibrium shape of the ground water mound for the average infiltration rate in the basins was determined according to the analog procedure presented in an earlier paper (3). In this analysis, the flux at the water table mound beneath the basins was considered uniformly distributed and taken as the average infiltration rate for the gross area of the recharge basins, which includes the dry strips between the basins. Since the width of each basin was 20 ft, the average rate for the six basins was calculated as 120 x 3.5/220 = 1.91 ft/day, where the number 3.5 indicates the average infiltration rate for the six recharge basins. The electrical current in the analog model at pseudo-equilibrium corresponded to the average recharge rate of 1.91 ft/day. Thus, the K-components could be calculated from the horizontal and vertical resistance values in the analog model in accordance with the procedure in (6). The resulting values were K_h = 282 ft/day and K_v = 17.6 ft/day.

Equipotentials and streamlines for the flow system at pseudo-equilibrium with an average infiltration rate of 1.91 ft/day are shown in Figure 16. Points A and B in this system refer to the bottoms of the 30-ft ECW and the 100-ft WCW, respectively. The potential at these points in the analog model indicated water level rises of 2.62 ft in ECW and 0.78 ft in WCW. These values closely agree with the actual rises of 2.7 ft and 0.7 ft observed in the field. Thus, the 16-fold ratio between horizontal and vertical conductivity evaluated for the horizontal water table assumed in the first analysis, and the resulting values of K_h and K_v, are valid.

Field Studies. The values of K_h and K_v were also evaluated from field measurements of K at the observation wells. The hydraulic conductivity K of the material at the bottom of each observation well was determined with the tube-method developed by Frevert and Kirkham (9). With this method, the water level in the well is lowered a few feet below its equilibrium position and the subsequent rate of rise is measured. The value of K is then calculated from the rate of rise and a factor expressing the geometry of the flow system. The results (Table 3) show that a wide range of values was obtained.

Previous measurements with the double-tube method in the Salt River bed indicated that K of the sandy material usually ranges from 1.7 ft/day to 30 ft/day, depending on the texture of the sand (5 and references therein). Thus, the values of K in Table 3 that are 34 ft/day or less apparently belong to sandy materials, whereas those of 173 ft/day and more would pertain to the more gravelly materials. The average of the values of 34 ft/day or less in Table 3 is 12.2 ft/day, which can be considered as the average K of the sandy materials. Similarly, the average K of the gravelly materials can be calculated as 458 ft/day.

Table 3. Results of hydraulic conductivity measurements.

Well	Depth, in feet	K, in feet per day
1	20	2.5
1-2	20	173
ECW	30	34
WCW	100	270
5-6	20	0.1 (estimated)
7	20	775
8	20	620

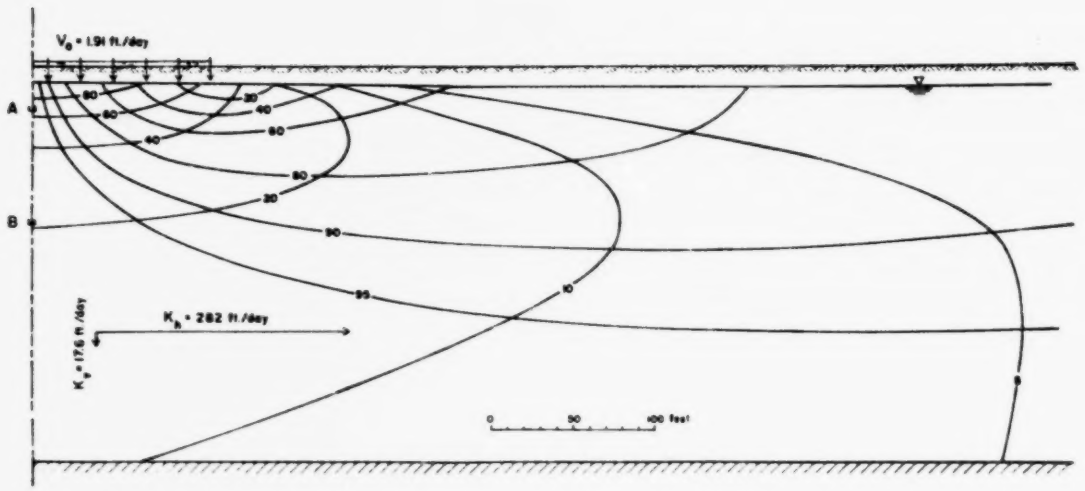

Figure 16. Ground water flow system at steady-state during recharge.

Assuming that the geologic profile consists of a regular succession of equally thick sandy and gravelly layers, the hydraulic conductivity in horizontal direction can be calculated as the arithmetic mean of the K-values of the two materials, and the hydraulic conductivity in vertical direction as the harmonic mean of these two values (7). Since the average value of K for the sandy materials is 12.2 ft/day, and that for the gravelly material 458 ft/day, this procedure yields K_h = 236 ft/day and K_v = 23.7 ft/day. This is in good agreement with the values of 282 ft/day and 17.6 ft/day, respectively, obtained with the analog procedure, despite the fact that replacement of the actual profile by a profile of alternating sand and gravel layers of equal thickness is a drastic simplification.

The ratio K_h/K_v calculated from the field measurements is lower than the ratio obtained with the analog (10 versus 16). This may be due to the fact that in the calculation of K_h and K_v from the hydraulic conductivity measurements at the observation wells the individual layers were considered isotropic. In actuality, however, each layer may be anisotropic in itself due to particle orientation and microstratification. This anisotropy was demonstrated by double-tube measurements on sandy deposits in the Salt River bed, which showed that although the sand was seemingly uniform, the hydraulic conductivity in horizontal direction was 7 times that in vertical direction (5). Nevertheless, the good agreement between the values of K_h and K_v evaluated with the analog and with the field measurements, which are two completely independent and widely different techniques, lends validity to the results of both.

The flow system below the water table shown in Figure 16 was used to predict underground detention times of the renovated water pumped from ECW and WCW (see next section). The flow system of Figure 16 was also used to evaluate the effective transmissibility of the aquifer for ground water recharge. This effective transmissibility was then used in an analog model of an operational system to predict water table positions and underground detention times for various designs (See VIII. DESIGN AND OPERATION OF LARGE-SCALE SYSTEM).

2. Underground Detention Times

The hydraulic gradients along the streamlines in Figure 16 can be used to predict underground velocities, and hence travel times, of the renovated water. This procedure was applied to the vertically downward streamline in the center of the system, which is the symmetry line of Figure 16. The hydraulic conductivity along this line is 17.6 ft/day.

The underground detention time is inversely proportional to the infiltration rate and directly proportional to the porosity of the soil material. Assuming an infiltration rate of 1 ft/day, and a porosity range of 20% to 30%, the calculations yielded travel times below the water table of 9.6 to 14.4 days for the water pumped from ECW (point A in Figure 16), and 189 to 284 days for that pumped from WCW (point B in Figure 16). The time for the water to travel from the

bottom of the basins to the water table is about 2 days per 1 ft/day infiltration rate. This time must be added to the travel times below the water table to get the total underground detention time.

In calculating the underground detention times from the gradients and hydraulic conductivity in the flow system of Figure 16, the assumptions were made that (1) the aquifer is uniformly anisotropic (thus ignoring the effect of any distinct layers of low permeability that may have an overriding effect on the travel time), (2) the difference in density between the reclaimed water and the more saline native ground water has no effect on the flow system, and (3) dispersion can be ignored and the native ground water is displaced in piston-like manner.

The calculated underground detention times can be checked by observing the arrival of nitrate peaks in the renovated water from the observation wells. The nitrogen in the sewage water held as capillary water in the upper portion of the profile during the dry-up period is completely nitrified. When a new inundation period is started, this nitrified capillary water is pushed ahead of the newly infiltrated water, and when it arrives at the intake of an observation well it causes temporarily a high nitrate concentration, or nitrate peak, in the renovated water.

The underground detention times can also be checked by monitoring the salinity level of the observation wells. Since the native ground water has a salinity that is 2 to 4 times as high as that of the sewage water, a reduction in the salinity of the well water from a few thousand parts per million to about 1000 parts per million indicates that native ground water has been displaced by renovated sewage water.

The nitrate-peak technique was used for determining underground detention times for the water from ECW. Table 4 shows how many days after the start of a new inundation period nitrate peaks were observed in ECW in 1969. The average infiltration rates during those days are shown in the third column. The last column of Table 4 shows the underground detention time as if the infiltration rate had been 1 ft/day. The average of the underground detention times per unit infiltration rate, i. e., 11.5, is close to the predicted value of 11.6 days for 20% porosity.

Since the calculated underground travel time for the water from WCW is 191-286 days (adding 2 days for travel from the basin bottom to the water table) for an infiltration rate of 1 ft/day, displacement of the native ground water by renovated sewage water at the bottom of the well should take place after 191 to 286 ft of water have infiltrated in the basins. At the end of July 1969, the salt content of WCW-water started a gradual drop from its 3000-4000 ppm range, indicating that renovated water had arrived at the bottom of the well. The average accumulated infiltration for the six basins since the start of the project was 315 ft on 19 July 1969. This is higher than the predicted range of 191-286 ft, but considering the simplifications and assumptions made in the calculations, the agreement is still reasonable.

Table 4. Occurrence of distinct nitrate peaks in ECW in relation to start of inundation period and average infiltration rate.

Day NO_3- peak was observed	Number of days after start of inundation	Average infiltration rate, ft/day		Number of days times infiltration rate
17 Jan 1969	11	0.83		9.1
12 Feb 1969	9	1.12		10.1
13 Mar 1969	10	1.00		10.0
25 Sep 1969	8	1.65		13.2
21 Oct 1969	6	1.98		11.9
26 Nov 1969	5	2.50		12.5
30 Dec 1969	7	1.94		13.6
			Av.	11.5

The long underground detention time of almost 2 years for the renovated water from WCW is due to the anisotropic nature of the aquifer, which offers much more resistance to water movement in vertical direction than in horizontal direction.

The observation wells 1, 1-2, 5-6, 7, and 8 were installed in April 1968. At that time, these wells were already yielding renovated sewage water, with the exception of well 8. The underground detention time for the water from wells 1-2 and 5-6 is about half that for ECW. The underground detention time for the water from wells 1 and 7 is 1 to 2 months, as deduced from the nitrate behavior in these wells. Well 8 continued to yield native ground water until January 1970, when the salt content started to drop from about 2000 ppm to about 1500 ppm. Thus, the underground detention time for renovated sewage water from well 8 is almost 2 1/2 years.

3. Water Table Response

Profiles of water levels in the observation wells are shown in Figure 17, which applies to the inundation period 21 November-8 December 1969. Since wells 1, 1-2, 5-6, 7, and 8 are all 20 ft deep, the water level readings from the 30-ft-deep ECW were corrected so that they would also apply to the 20-ft depth. This was done using the vertical gradient in the center of the flow system as evaluated by analog (Figure 16). Thus, the water levels as shown in Figure 17 all apply to the piezometric head at 20-ft depth, which is about 10 ft below the water table.

The water level profile on 21 November, prior to the inundation period, is essentially horizontal, indicating static ground water conditions in the north-south direction. The horizontal profile also indicated negligible effect of seepage from the effluent stream north of the project area (Figure 1) on the ground water flow system. The water level in the stream is about 2 1/2 ft above the static ground water level. The profile on 1 December shows the water levels at their highest positions, which is also the pseudo-equilibrium level for the first part of the inundation period. Because basins 3 and 4 exhibited the highest infiltration rates of this period, the water level in ECW is probably higher than it would have been in case the infiltration rates had been the same for all basins. The profile on 8 December shows the water levels in the wells at the end of the inundation period, after the water depth in the recharge basins was changed from 13 to 7 inches on 4 December. On 11 December, 3 days after the flow into the basins was stopped, the water level profile in the observation wells was essentially horizontal, and on 15 December it had returned to about the same static position as before the start of the inundation period.

Comparing the profiles on 1 and 8 December shows that the water level in well 5-6 is slower to rise and slower to fall than, for example, the water level in well 1-2. This slower response of well 5-6 is due to the materials of relatively low hydraulic conductivity around this well (Table 3).

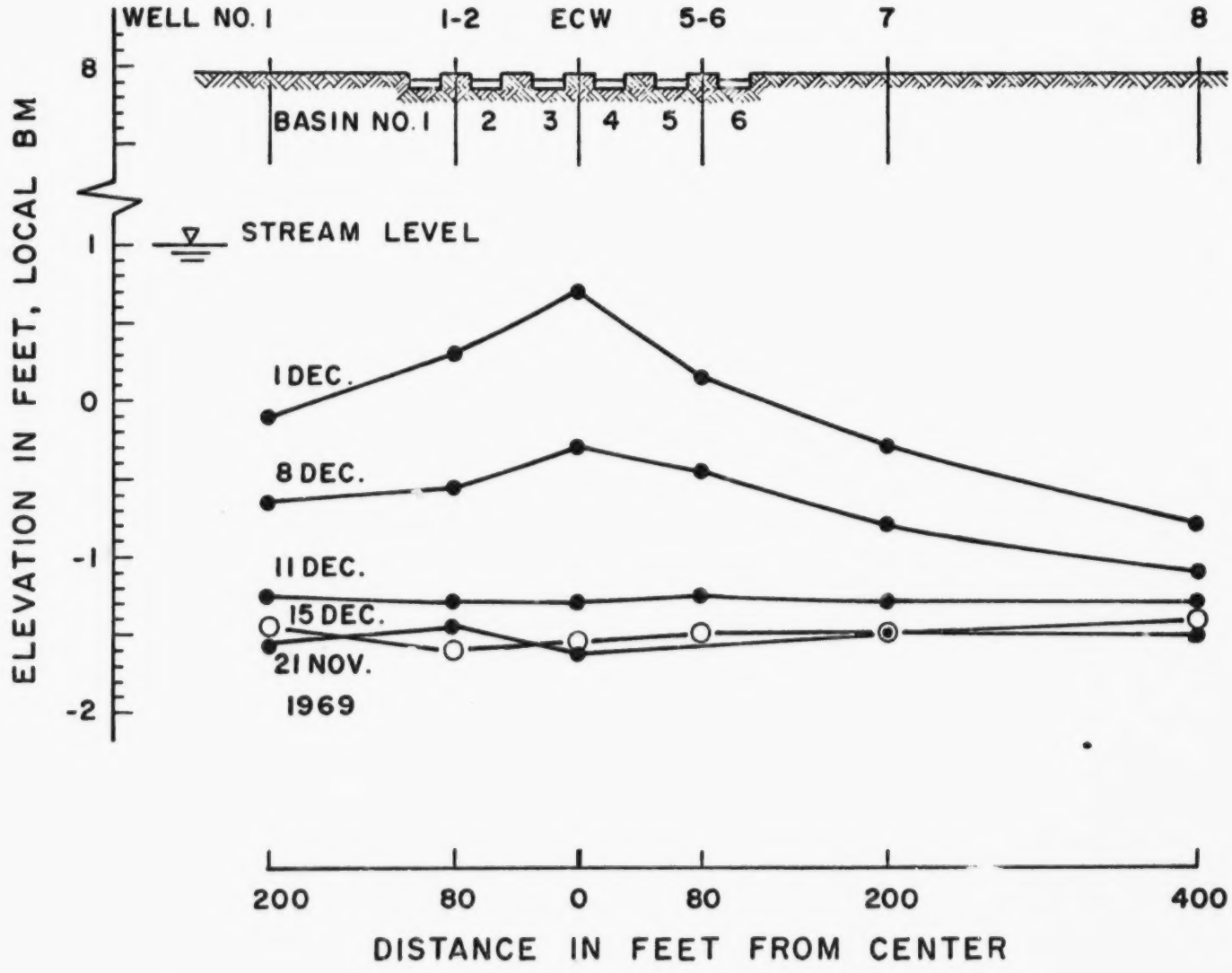

Figure 17. Water-level profiles in observation wells during inundation and dry up.

The rise of the water levels to semi-equilibrium positions and the relatively rapid decline to the original static levels after inundation is stopped are typical for the Flushing Meadows conditions where the aquifer is of considerable lateral extent and of high hydraulic conductivity in horizontal direction.

4. Effect of Recharge on Hydraulic Conductivity of Aquifer

Since the water table rises during infiltration were small compared to the total height of the flow system, the water level rises in the observation wells at pseudo-equilibrium conditions should be in direct proportion to the infiltration rate in the basins. Thus, the water level rise per unit infiltration rate should remain constant if the hydraulic conductivity of the aquifer does not change. An increase in the water level rise per unit infiltration rate would indicate a decrease in the hydraulic conductivity of the aquifer, and vice versa.

To study the possibility of a change in the hydraulic conductivity of the aquifer, the water level rise at pseudo-equilibrium in ECW was measured each fall and divided by the average infiltration rate in the basins for the flooding period in question. The following results were obtained

	Static water level in ECW prior to recharge ft local BM	Equilibrium rise per unit infiltration rate $ft/_{ft/day}$
September 1967	- 6.0	0.77
October 1968	- 2.0	0.71
November 1969	- 1.5	0.72

These data indicate that from 1967 to 1968, the hydraulic conductivity of the aquifer actually increased. This increase may, at least in part, be due to the 4-ft rise in the static ground water level from 1967 to 1968, which could have included some permeable strata into the aquifer. From 1968 to 1969, the hydraulic conductivity of the aquifer remained essentially constant.

SECTION VII

WATER QUALITY STUDIES

Composite samples of the secondary sewage effluent, which was the influent for the Flushing Meadows Project, were obtained almost daily with a continuous sampler placed on the head box at the upper end of one of the basins (Figure 1). The sampler was of the dipstick type, taking approximately 0.5 ml effluent every minute and yielding about 0.7 liter (about 1/5 gallon) per 24 hrs. The bottle collecting the sample was in a thermally insulated box cooled with icepacks.

Grab samples were obtained from the observation wells. The wells were bailed or pumped several times before collecting the sample to make sure that a fresh sample was obtained. The wells within the basin area (1-2, ECW, and 5-6, Figure 1) were usually sampled daily. The other wells were sampled weekly, monthly, or bimonthly. Well 1-2 yielded water that had mainly infiltrated in basins 1 and 2. The water from ECW came essentially from basins 3 and 4, and water from well 5 from basins 5 and 6, provided that all basins were inundated at the same time.

All samples were transported to the U. S. Water Conservation Laboratory, a distance of about 25 miles, and kept under refrigeration until analysis. The chemical and bacteriological analyses were performed using the procedures of Standard Methods (11) as a guide. Although analyses were started in 1967, the analytical laboratory did not function smoothly until 1968. Therefore, results obtained after 1 January 1968 will be reported only.

1. Biochemical and Chemical Oxygen Demand

The BOD_5 of the secondary effluent was usually in the 10-20 ppm range. After traveling through 30 ft of sand and gravel (10 ft above and 20 ft below the water table), the BOD_5 of the renovated water, as sampled from ECW, ranged from 0 to 1.2 ppm in 1968 with an average of about 0.3 ppm. In 1969, the BOD_5 of the samples from ECW did not exceed 2 ppm and was less than 0.5 ppm for the summer months (Figure 19). The BOD-values of the renovated sewage water were determined by the laboratory of the 91st Avenue Phoenix Sewage Treatment Plant. Although the BOD_5 of the renovated water was essentially zero, the organic carbon content of the renovated water from ECW was in the 2-10 ppm range, as was determined later with the total carbon analyzer (Beckman model 915).

The chemical oxygen demand was determined with the dichromate technique. The COD of the effluent was determined on the supernatant liquid in the sample bottle so that it would be characteristic of the effluent as it would move into the soil, relatively free of suspended material. The COD of the effluent usually was in the 30-60 ppm range with the values in the first half of the year being somewhat higher than in the second

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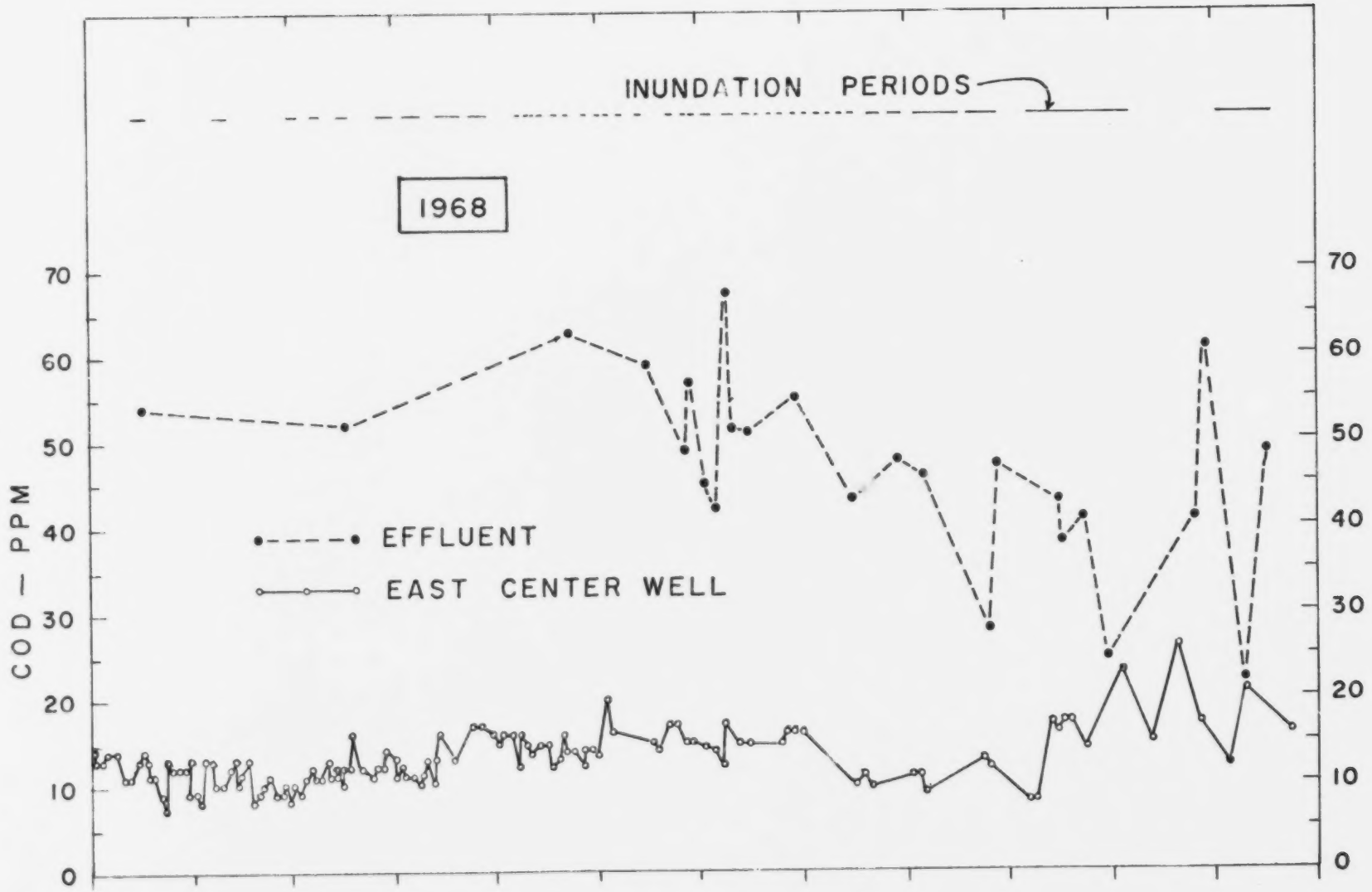

Figure 18. COD of secondary effluent and of renovated water from East Center Well in 1968.

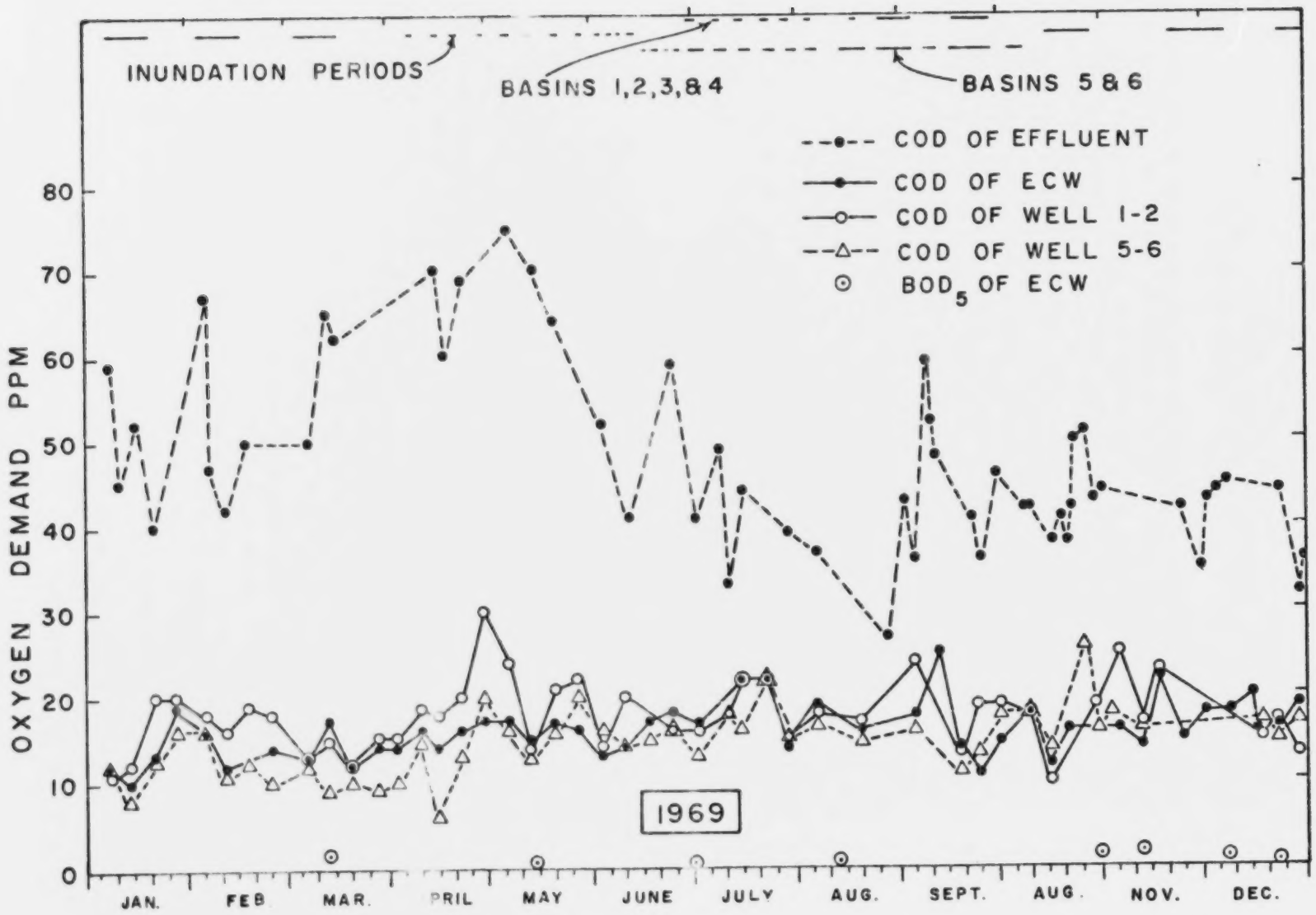

Figure 19. COD of secondary effluent and of renovated water from ECW, Well 1-2, and Well 5-6 in 1969.

half of the year (Figures 18 and 19). This is probably due to the poorer quality of the effluent in winter and spring.

The COD of the renovated water from ECW was generally in the 10-20 ppm range (Figures 18 and 19). The length of the inundation period and the surface condition of the basins (grass, gravel and bare soil) apparently had no significant effect on the COD of the renovated water. Figure 19 shows that the COD of the renovated water from well 1-2 was slightly higher than that of ECW. This may be due to the fact that well 1-2 is only 20 ft deep, compared to 30 ft for ECW. The COD of the water from well 5-6 was slightly lower than that of ECW, which could be due to the finer-textured soil around the bottom of this well.

The COD of the renovated water from wells 1, 7, 8, and EW, was about the same as that for wells 1-2, ECW, and 5-6, i. e., an average of 14 ppm (Table 5). Thus, the additional underground travel of the renovated water to the "outlying" wells had little or no effect on COD.

The COD of the native ground water was slightly higher than that of the renovated sewage water, i. e., an average of about 20 ppm in 1969 for the three wells yielding native ground water (Table 5). The 91st Avenue well mentioned in this table is an irrigation well located about 1 1/2 miles east of the Flushing Meadows Project. The well pumps from a depth of about 100 to 200 ft.

2. Nitrogen

Nitrogen content of the water samples was determined with the brucine method for nitrate, the diazotization method for nitrite, the Nessler technique for ammonium (later supplanted by the distillation technique), and the Kjeldahl method for organic nitrogen.

The total nitrogen content of the effluent generally ranged from 20 to 35 ppm with the concentrations in the summer being somewhat lower than in the winter (Figures 20 and 21). Almost all the nitrogen in the effluent was in the ammonium form. The organic nitrogen content of the effluent was about 1 ppm (about 3 ppm when the suspended solids content was high), and the nitrate-nitrogen concentration was about 0.1 ppm.

If short, frequent inundation periods were used (for example, 2 or 3 days wet, 2 to 5 days dry), almost all the nitrogen of the effluent was converted to the nitrate form in the renovated water (Figure 20). If longer flooding periods were used (for example, 2 to 4 weeks wet and 1 to 2 weeks dry), the nitrate-nitrogen concentration in the renovated water was considerably lower (April-May and September-December, Figure 20), except for distinct "peaks." These nitrate peaks occurred 5 to 10 days after the start of a new inundation period and they are apparently caused by the arrival of nitrified effluent water that was held as capillary water in the soil during the preceding dry-up period.

Table 5. COD in mg/liter for various wells.

Date	1[a]	7[a]	EW[a]	WCW[b]	8[b]	91st Avenue[b]
1968	10-12	12-16		11-23	10-12	10-17
23 Jan 1969	12	12		31	11	
20 Feb 1969	12	10		22	12	24
12 Mar 1969	12	16		19	16	14
26 Mar 1969	6	6		24	8	14
2 May 1969	12	7		20	7	11
6 Jun 1969	17	12			8	
16 Jul 1969		16		35	27	
15 Aug 1969	16	12		37	22	
10 Sep 1969	25	25		29	30	
10 Oct 1969	23	18		29	26	29
14 Nov 1969	16	16	15	21	20	
18 Dec 1969	8	13	13	13	17	21
Av. 1969	14	14	14	25	17	19

[a] renovated sewage water

[b] native ground water

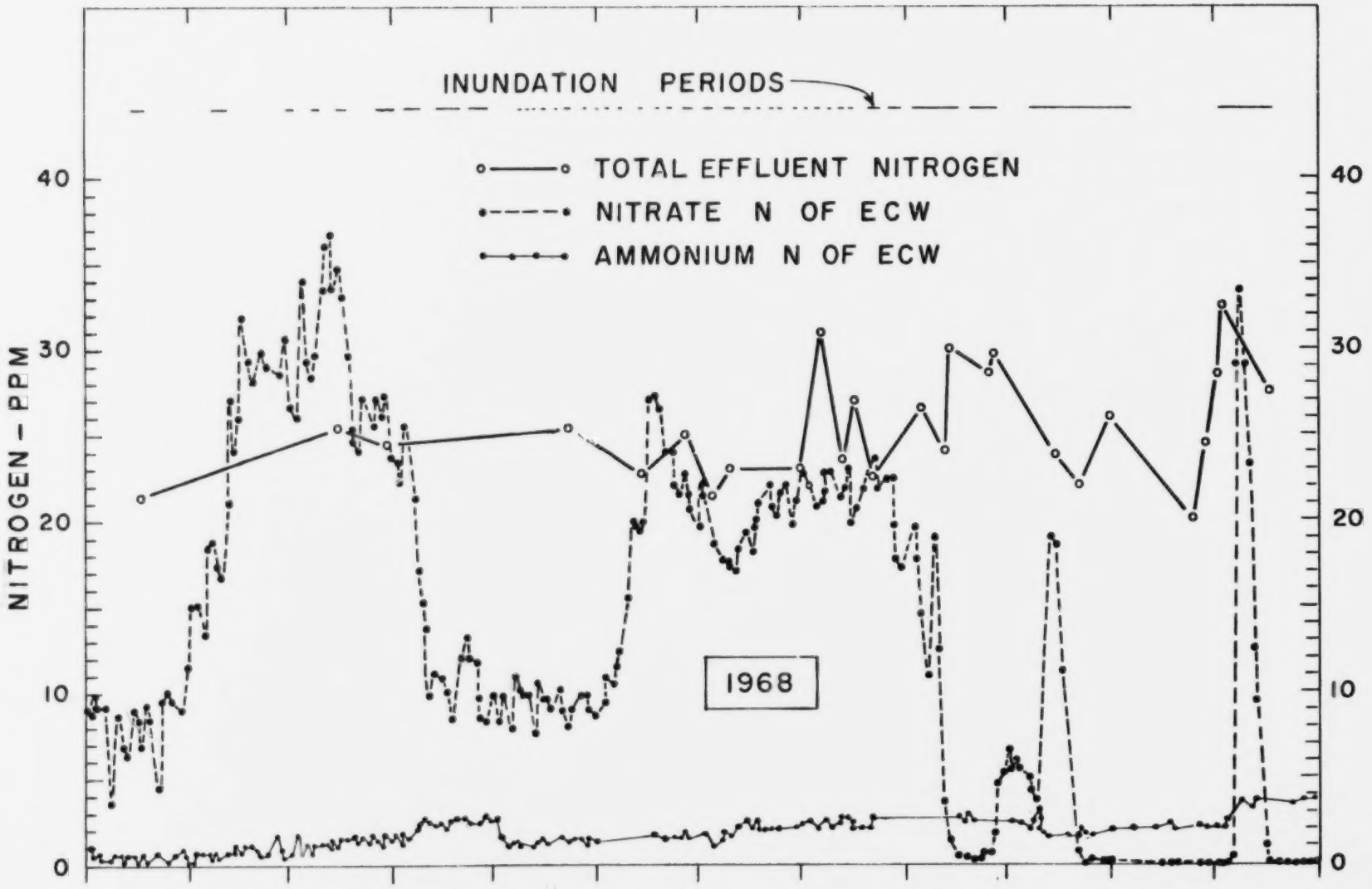

Figure 20. Total nitrogen of secondary effluent, and nitrate N and ammonium N in renovated water from ECW in 1968.

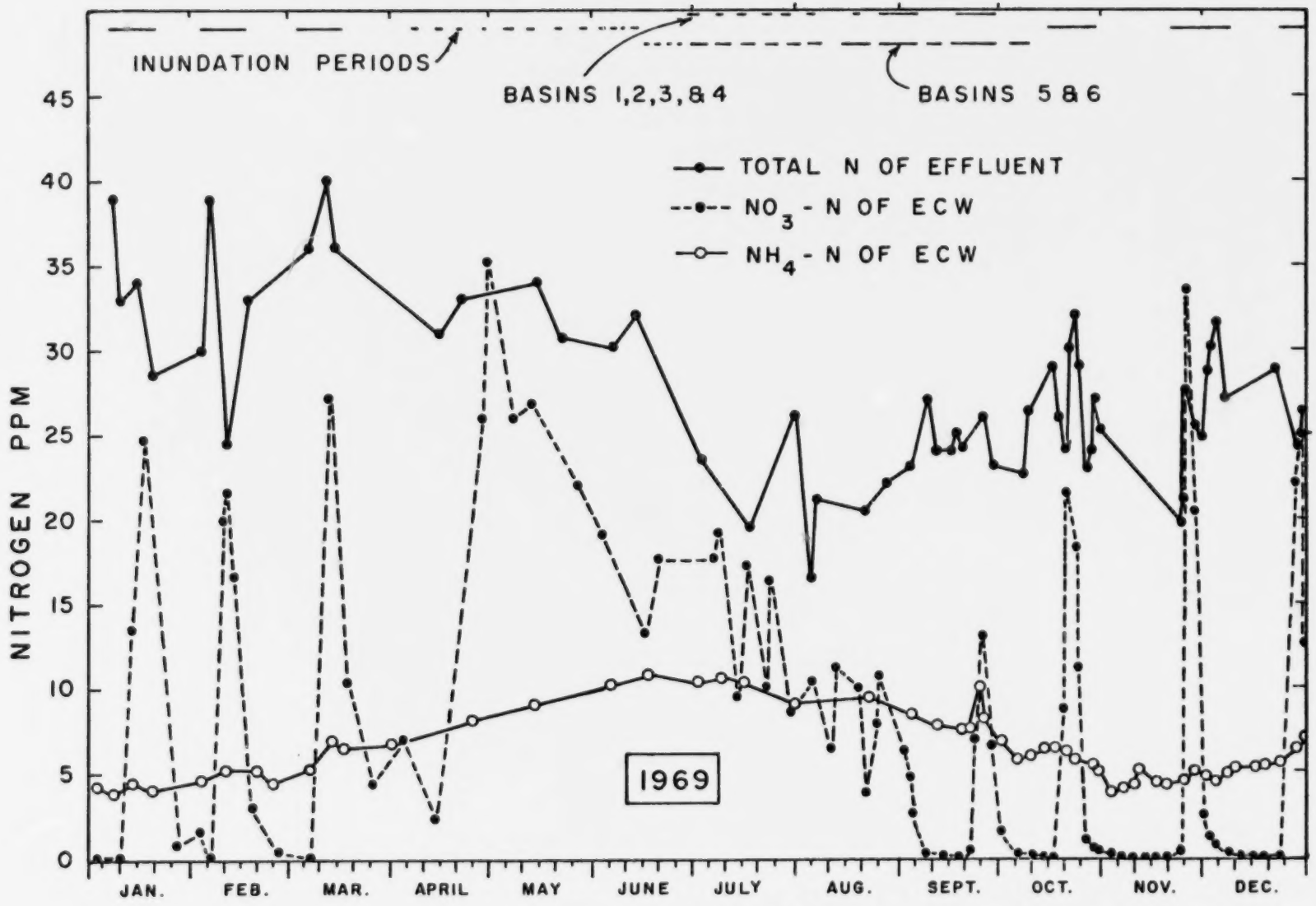

Figure 21. Total nitrogen of secondary effluent, and nitrate N and ammonium N in renovated water from ECW in 1969.

Because of the aerobic conditions in the upper soil layers during dry up, the ammonium nitrogen in the effluent and the ammonium adsorbed to the soil and organic matter could be converted to nitrate. When a new flooding period was started, the nitrified capillary water was then pushed downward by the newly infiltrating water. Arrival of this nitrified water at the intake of an observation well caused a nitrate peak in the water sampled from that well.

The nitrate behavior in the renovated water from ECW in 1969 was similar to that in 1968 (Figure 21). During the short, frequent inundation periods from April to about July, most of the nitrogen was converted to the nitrate form. With the long inundation periods in the first and last parts of the year, nitrate-nitrogen contents were close to zero, except for the peaks which again occurred 5 to 10 days after the start of the inundation periods.

Nitrogen can be removed from waste water moving through soil by various processes, including adsorption of ammonium to the clay and organic fraction of the soil, fixation of ammonium by the organic fraction, fixation of nitrogen in tissue of soil microorganisms, nitrogen uptake by vegetation, volatilization of ammonia, and denitrification. Of these processes, only uptake of nitrogen by crops (if the crop is harvested), volatilization of ammonia, and denitrification cause a net removal of nitrogen. The other processes merely store nitrogen in the soil.

Quantitative analysis of the nitrogen-removing capability of each of these processes at the Flushing Meadows Project indicated that the sustained removal of nitrogen observed at the project must mainly be attributed to denitrification. This process requires the simultaneous occurrence of nitrates and organic carbon under anaerobic conditions. The main end product of denitrification is free nitrogen gas, which escapes to the atmosphere or dissolves in the downward moving water.

That storage of nitrogen in the soil was insignificant became evident at the end of 1969, when the soil of the recharge basins was analyzed for total nitrogen. The analyses showed that the loamy sand contained only about 0.1 mg nitrogen per gram of dry soil more than the virgin soil outside the recharge basin. This nitrogen could account for only a small fraction of the nitrogen removed from the 450 ft of sewage effluent which the recharge basins had received by the end of 1969.

As regards the mechanism of nitrogen removal by denitrification, when a new flooding period is started, there is entrapped air in the soil for nitrification of ammonium during the initial part of the flooding period. As the oxygen will be consumed, however, an aerobic pocket will begin to develop where nitrate and organic carbon can both be present, creating conditions favorable for denitrification. With continued flooding, all oxygen will eventually be used up and the nitrogen of the effluent will stay in the ammonium form, which can be adsorbed by the clay and organic fraction of the soil. If the inundation is not stopped before the cation exchange complex in the soil is

saturated with ammonium, increased ammonium levels in the renovated water can be expected. When the inundation is stopped, air will enter the soil and the resulting aerobic conditions in the upper soil layers will enable nitrification of the adsorbed ammonium. Some of the nitrates formed in this process may diffuse to anaerobic micro-environments in the same soil and denitrification can occur if organic carbon is also present. Some of these nitrate ions may also mix later with the newly infiltrating water when a new flooding period is started and move down to anaerobic environments where denitrification may occur.

Conditions for denitrification tend to be more favorable in root zones than in soils without roots (13). This is because roots contribute organic carbon to the soil by direct exudation from living roots and decomposition of dead roots. Also, the oxygen uptake by the roots will help to create anaerobic environments necessary for denitrification.

Evidence that denitrification is more complete below the vegetated basins than below the nonvegetated basins of the Flushing Meadows Project is presented in Figure 20. This figure shows that the nitrate-nitrogen levels in the renovated water from ECW during long flooding periods were lower in the fall when the bermudagrass was fully developed, than in April and May when basins 3 and 4 from which ECW mainly receives its water were still bare. Also, the nitrate-nitrogen levels in the water from well 1-2, which receives water from the nonvegetated basins 1 and 2, were higher in the fall of 1968 than the nitrogen levels in the water from ECW and well 5-6 (Figure 22). Well 5-6 receives its water mainly from basins 5 and 6, which were also seeded to bermudagrass in the spring of 1968. Thus, nitrogen removal from waste water seems to be greater below vegetated basins than below nonvegetated basins.

In 1969, the nitrate peaks and nitrate levels between peaks of the renovated water from well 1-2 were higher than those from ECW and well 5-6 in the beginning of the year (Figure 23). However, later in the year, when basin 1 was covered with a full stand of native grasses, the nitrate peaks and the nitrate concentrations between the peaks for well 1-2 were more in line with those for ECW and well 5-6. This is another indication that the development of vegetation may have contributed to increased denitrification in the soil.

Since direct uptake of nitrogen by the plant roots is insignificant when compared to the total nitrogen loading of the recharge basins, which is about 100 lbs of nitrogen per acre per day during flooding, the lower nitrate-nitrogen levels in the renovated water below vegetated basins must be attributed to more complete denitrification in the root zone.

The ammonium nitrogen concentrations in the renovated water were initially low but gradually increased until about July 1969

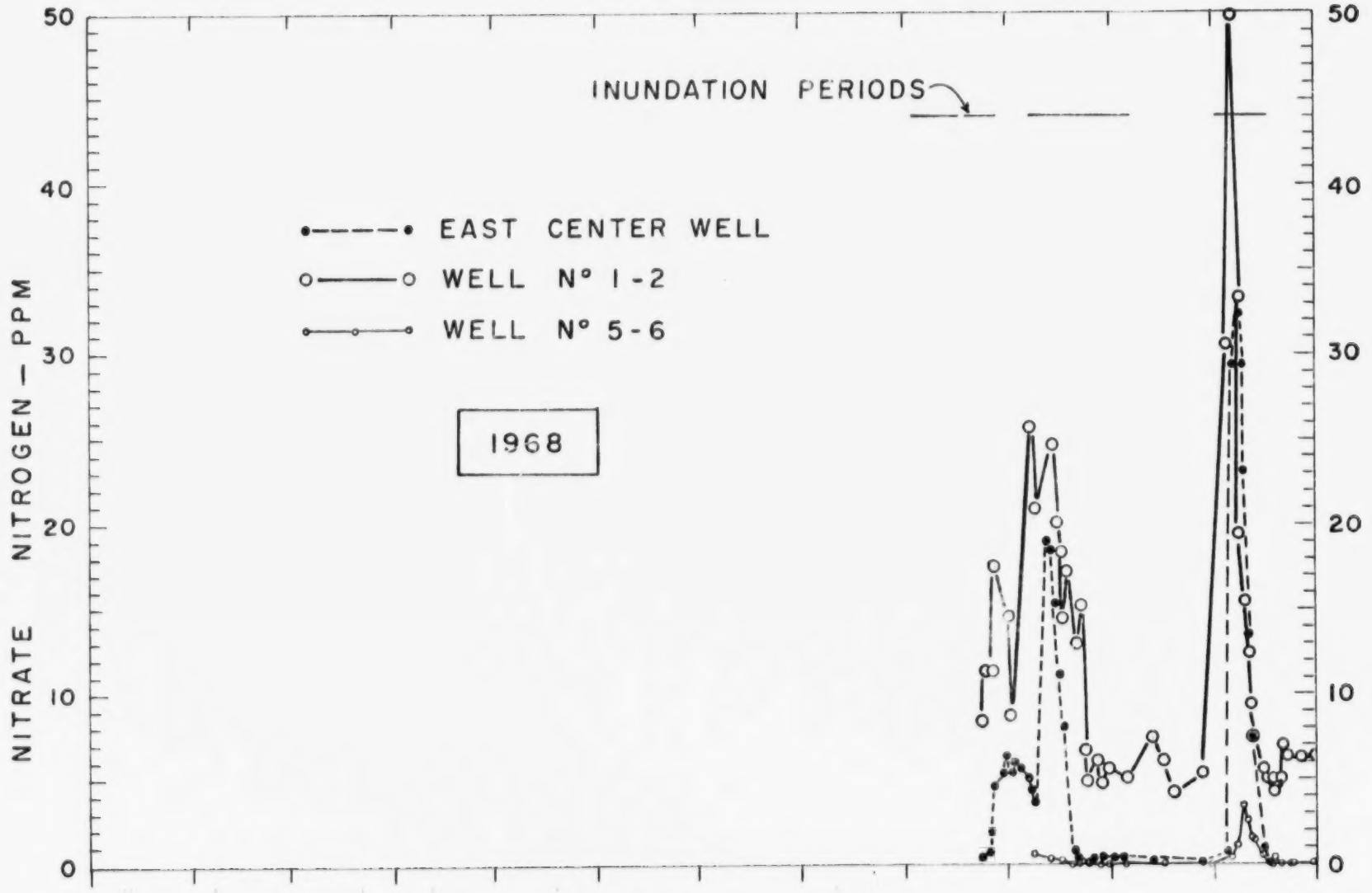

Figure 22. Nitrate N in renovated water from wells 1-2, ECW, and 5-6 in 1968.

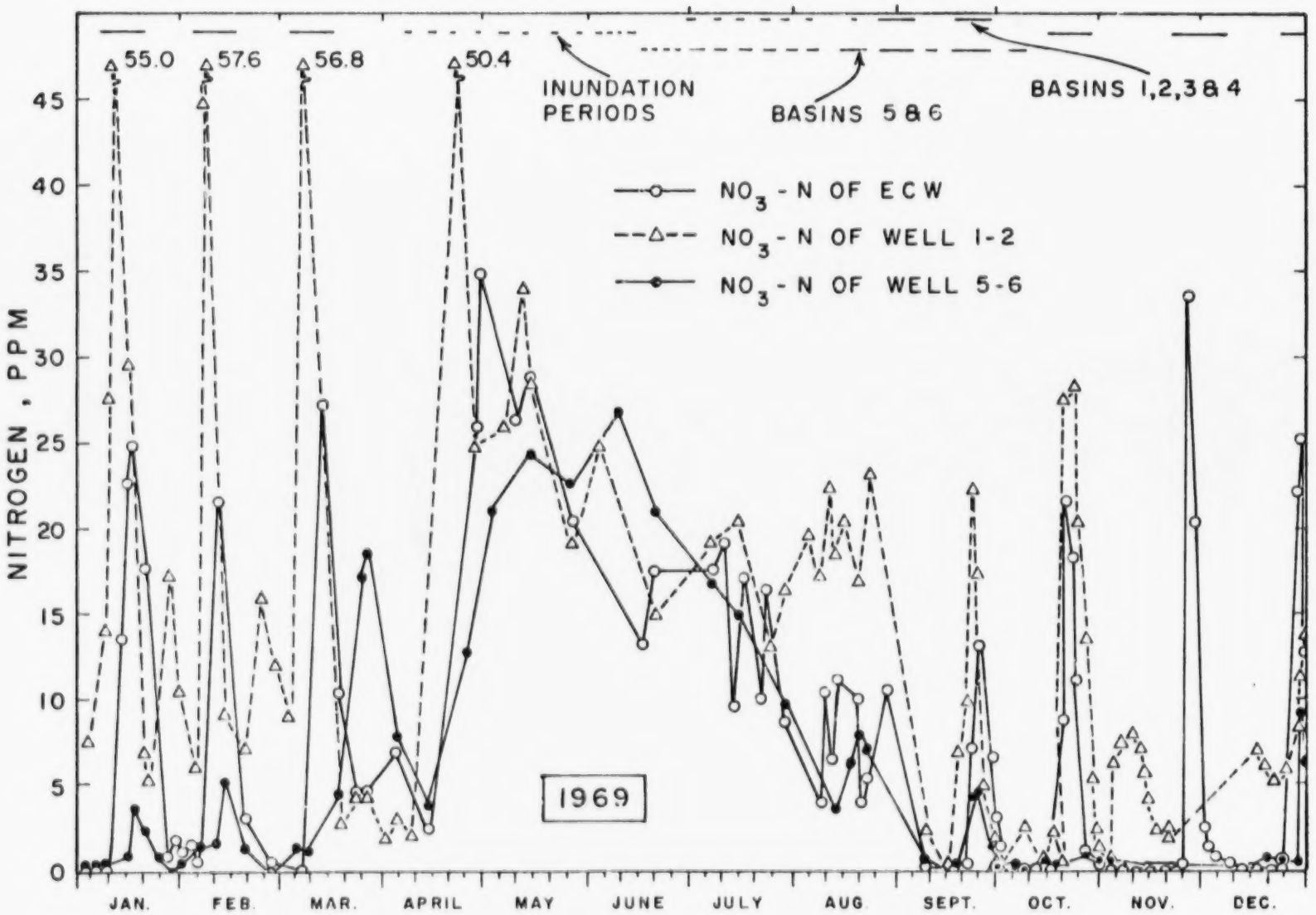

Figure 23. Nitrate N in renovated water from wells 1-2, ECW, and 5-6 in 1969.

(Figures 20 and 21). This increase was apparently due to saturation of the cation exchange complex in the soil with ammonium, particularly during sequences of long flooding periods. A sequence of short flooding periods should restore the capacity of the soil to adsorb ammonium, because the predominantly aerobic conditions in the upper layers of the soil during short flooding sequences would favor nitrification of the adsorbed ammonium. That this may indeed occur is evidenced in Figure 21, which shows that when the sequence of long flooding periods used from September 1968 to April 1969 was changed to a sequence of short flooding periods, the ammonium nitrogen concentrations in the renovated water from ECW began to decrease in June 1969 (the delay is probably due to the underground detention time and the time necessary for a population of nitrifying bacteria to develop). Another factor that may have contributed to the 1969 summer reversal of the upward trend in the ammonium nitrogen concentration of the ECW water is that in the summer, drying conditions are better and the upper soil layers have a lower water content than in the winter. This lower water content causes an increase in the oxygen diffusion rate and hence in the depth to which the soil profile becomes aerobic during dry-up periods. Thus, adsorbed ammonium can be nitrified to greater depths and the adsorptive capacity for ammonium can be better restored in the summer.

Nitrate and ammonium nitrogen concentrations for the wells outside the basin area are shown in Tables 6 and 7, respectively. The nitrate-nitrogen levels in the renovated sewage water from the outlying wells also show the effect of the length of the inundation periods. The change to longer inundation periods in July 1969, for example, must have been responsible for the drop in nitrate levels in the renovated water from wells 1 and 7 in August and September 1969, respectively. Ammonium levels in the renovated water from wells 1 and 7 were lower than those for the wells inside the basin area, indicating additional removal of ammonium as the renovated water travelled laterally beneath the water table.

Organic nitrogen in the renovated water was about 0.9 ppm or less for the wells inside the basin area, and about 0.5 ppm for wells 1 and 7. Nitrite-nitrogen concentrations in the renovated sewage water were generally in the 0-1 ppm range, with the values being close to zero most of the time.

The results of the nitrogen studies show that the form and concentration of the nitrogen in the renovated water can be controlled by the length of the inundation period and, to a lesser extent, by the use of vegetation in the basins. If short inundation periods of a few days each are used, almost all the nitrogen in the sewage effluent is converted to the nitrate form in the renovated water. With very long inundation periods, for example, several months, the nitrogen will mostly stay in the ammonium form. With inundation periods of a few weeks, almost zero nitrate-nitrogen concentrations can be expected in the renovated water, after distinct nitrate peaks which occur when nitrified effluent held

Table 6. Nitrate nitrogen concentrations in mg N per liter for various wells.

Date	1[a]	7[a]	8[b]	EW[a]	WCW[b]	91st Avenue[b]
1968	0.2-16	1.3-3.2	0.2-4.2		0-7.6	6.4-6.7
23 Jan 1969	14.2	7.6	2.0		3.5	6.0
20 Feb 1969	16.0	15.6	2.1		4.1	
12 Mar 1969	14.0	18.4	0.3		4.7	5.8
26 Mar 1969	11.5	11.8	0.3		1.8	5.9
2 May 1969	23.2	4.2	0.2		2.2	4.3
6 Jun 1969	21.6	10.8	0.2		2.8	
16 Jul 1969	22.4	17.6	0.1		3.5	5.4
15 Aug 1969	2.0	14.8	0.5		3.5	
10 Sep 1969	1.3	5.0	0.3		3.3	
10 Oct 1969	0.3	3.4	0.1		2.5	6.4
14 Nov 1969	0.2	0.0	0.3		3.7	
18 Dec 1969	0.2	0.5	0.5		4.4	5.5
28 Nov 1969				8.5		
3 Dec 1969				6.4		
9 Dec 1969				3.3		
17 Dec 1969				0.2		
22 Dec 1969				4.8		
29 Dec 1969				8.3		

[a] renovated sewage water

[b] native ground water

Table 7. Ammonium nitrogen concentrations in mg N per liter for various wells.

Date	1[a]	7[a]	8[b]	EW[a]	WCW[b]	91st Avenue[b]
1968	0.8-2.9	0.2-0.5	0.1-0.5		0-0.6	0-0.6
23 Jan 1969	3.3	1.1	0.2		0.1	0.1
20 Feb 1969	2.8	1.2	0.2		0.4	
12 Mar 1969	2.9	0.9	0.2		0.0	0.0
26 Mar 1969	2.8	0.7	0.1		0.0	0.1
2 May 1969	3.4	0.4	0.2		0.0	0.0
6 Jun 1969	2.8	1.3	0.0		0.0	
16 Jul 1969	2.9	0.2	0.0		0.5	
15 Aug 1969	2.0	1.2	0.4			
10 Sep 1969	3.8	1.5	0.1		0.0	
10 Oct 1969	4.1	1.5	0.1			0.0
14 Nov 1969	3.8		0.1		0.1	
18 Dec 1969	2.8	1.1	0.1		0.1	0.0
28 Nov 1969				2.2		
3 Dec 1969				2.8		
9 Dec 1969				3.6		
17 Dec 1969				4.9		
21 Dec 1969				3.5		
29 Dec 1969				5.6		

[a] renovated sewage water

[b] native ground water

as capillary water in the soil during dry up arrives at the intake of the well. Also, nitrogen removal is greater below vegetated basins than below nonvegetated basins, which is most likely due to increased denitrification in the root zone.

Nitrogen removals of 90% after the passage of the nitrate peak have been achieved. If the nitrate peaks are included, the total nitrogen removal is probably of the order of 30%. The remaining 70% of the nitrogen, however, is concentrated in the nitrate peaks which constitute only a small volume of the renovated water. This small volume could be recycled through the basins, it could be used where high nitrate levels in the water are not undesirable (for example, irrigation of parks or forage crops), or it could be pumped to a plant for denitrification with methanol or other carbon source.

Continued use of long inundation periods for nitrogen removal apparently causes the cation exchange capacity of the soil to become saturated with ammonium, after which the ammonium levels in the renovated water begin to increase. Additional lateral movement of the renovated water below the water table may be effective in removing some of this ammonium. However, it is probably more desirable to restore the nitrogen-removing capacity of the recharge system by changing to a sequence of short, frequent inundation periods. The resulting nitrification of the adsorbed ammonium then restores the ability of the cation exchange complex in the soil to adsorb ammonium. The same may be achieved by using some extra long dry-up periods in the summer so as to allow the soil to dry to lower water contents, which increases the oxygen diffusion rate and hence the depth of the aerobic zone. Growing a crop may also be effective in restoring the nitrogen-removing capability of the soil system, not only because the crop will remove nitrogen from the soil but the moisture uptake by the roots will increase the oxygen diffusion rate and hence the depth of the aerobic zone. The root zone could also contribute to denitrification.

3. Phosphate

Inorganic phosphorus in the secondary effluent and in the renovated water from the observation wells was mainly in the ortho-phosphate form and was analyzed with the Murphy-Riley technique. The results (Table 8) indicate phosphate-phosphorus concentrations of about 13 ppm in the effluent and essentially zero in the native ground water. When renovated sewage water arrived at the observation wells, the phosphate-phosphorus concentration increased to about 5-7 ppm, but remained relatively constant thereafter. Preliminary indications are that additional underground travel further reduces the phosphate-phosphorus concentrations, but time will tell how long the low phosphate levels in WCW and well 8, which began to yield renovated water after 1969, will continue.

The phosphate removal at the Flushing Meadows Project seems to occur gradually as the effluent water moves through the sands and gravels.

Table 8. Phosphate-phosphorus concentrations in mg P per liter of effluent, renovated water, and native ground water.

		Renovated water						Native ground water		
	Effluent	1-2	5-6	ECW	EW	1	7	8	WCW	91st Ave.
1968	8-18	5-7	0.1-0.5	2.4-5		0.05-0.15	0.5-0.8	0-0.1	0-0.2	
Feb 1969	14	8	0.1	7		1	2	0	0	0
May 1969	15	6.5	0.5	5		1.7	1	0.1	0	0.1
Aug 1969	12	3.5	1.2	4		2	4	0.05	0.1	0
Nov 1969	15	6.5	1.5	6	1.7	1	3	0.2	0.1	0.2

This is probably because of the low clay content and absence of iron and aluminum oxides in these materials. The removal of phosphate is probably due to precipitation of calcium-phosphate complexes, which are formed in the slightly alkaline and calcium-rich environment of the effluent water as it moves through the sands and gravels.

Phosphates are the only constituents of the effluent that precipitate in quantity in the soil. Thus, it would be of interest to calculate how this precipitate may affect the porosity, and hence the hydraulic conductivity, of the sands and gravels in the future. The phosphate will be assumed to precipitate as oxy-apatite, formula $3Ca_3(PO_4)_2.CAO$, of which the density is 3.2. Assuming that the amount of P precipitated in the soil is 10 mg/liter of effluent, that this precipitation takes place in a soil body 33 ft deep and 131 ft wide (considering two-dimensional flow only), and that the infiltration rate is 330 ft/year, the volume of apatite after 120 years of recharge would occupy about 0.5% of the total soil volume. At a pore space of 20%, the apatite would then occupy about 2.5% of the pore space. This is not likely to have a significant effect on the hydraulic conductivity. The above calculations are based on a very simplified process, but they show that it will probably take a very long time before phosphate accumulation in the sands and gravels will have an effect on the hydraulic performance of the system.

4. Boron

Boron concentrations, determined with the curcumin technique, were generally in the 0.4-0.5 ppm range for the secondary effluent as well as for the renovated water (Table 9). The boron concentration of the native ground water was also in about the same range.

Boron is not removed as the sewage water moves through the sands and gravels, indicating that such boron-fixing materials as iron and aluminum oxides are essentially absent. Boron concentrations of more than 0.5 ppm are undesirable for irrigation of boron-sensitive crops, such as citrus. With the increased use of low- or no-phosphate detergents, some of which contain borax or perborate, the boron concentration of the sewage effluent, and hence that of the renovated water, can be expected to increase in the future.

5. Fluoride

Fluoride concentrations, determined with the SPADN method, were about 4.5 ppm for the effluent (Table 10). This high concentration is probably due to certain industrial wastes. More than 50% of the fluoride ions are removed by the time the water has reached ECW, and additional removal takes place as the water travels laterally to the outlying well (Table 10). The fluoride removal parallels the phosphate removal, indicating that the fluorides may be precipitated with the calcium-phosphate complexes as fluorapatites.

Table 9. Boron concentration in mg B per liter for effluent, renovated water, and native ground water.

		Renovated water						Native ground water		
Date	Effl.	1	1-2	ECW	5-6	7	EW	8	WCW	91st Ave.
1968	0.43-0.50			0.43-0.56					0.63	0.64
23 Jan 1969	0.40	0.38	0.39	0.39		0.38		0.38	0.50	
26 Mar 1969		0.45				0.41		0.35		
25 Apr 1969	0.50		0.45	0.40	0.39					
16 Jul 1969						0.50				
15 Aug 1969	0.50	0.50	0.50	0.50	0.50			0.50		
14 Nov 1969		0.46	0.45	0.40		0.44			0.58	
18 Dec 1969	0.36		0.44	0.46	0.45	0.44	0.47			

Table 10. Fluoride concentration in mg F per liter for effluent, renovated water, and native ground water.

Date	Effl.	Renovated water						Native ground water		
		1	1-2	ECW	5-6	7	EW	8	WCW	91st Ave.
1968	5.0-5.5	1.2-1.7	2.1-2.8	2.1-2.5	1.8-2.3	1.8-1.9		0.8	0.8	
23 Jan 1969	2.7	1.3	3.5	1.9	1.4	2.0		0.8	0.8	
20 Feb 1969		1.3	3.1	2.1	1.6	1.9		0.8	0.8	0.6
26 Mar 1969		1.5	2.9	2.6	1.8	1.5		0.8	0.9	0.6
6 Jun 1969	5.6	1.5	2.6	2.3	1.8	1.2		0.6	0.8	
10 Sep 1969	4.3	2.1	2.4	2.6	2.6	2.7		0.8	0.8	
10 Oct 1969	5.3	2.0	2.3	2.6	2.6	2.9		0.7	0.6	0.6
14 Nov 1969	4.3	1.8	2.5	2.7	2.8	2.3		0.8	0.6	
18 Dec 1969	4.7	2.2	2.8	3.1	3.0	2.7	2.6	0.8	0.7	0.6
Av. 1969	4.5	1.7	2.8	2.5	2.2	2.1	2.6	0.76	0.75	0.6

6. Total Dissolved Salts

Total dissolved salts were determined by measuring the electrical conductivities of the water samples, and multiplying millimhos per centimeter by 640 to obtain ppm.

The salt concentration of the secondary effluent was about 1020 ppm, that of the renovated water about 1060 ppm, and that of the native ground water in the 2000-4000 ppm range (Table 11). The fact that the salt content of the renovated water was about 4% higher than that of the effluent may be largely due to evaporation from the water surface in the basins and from the soil during dry up. The evaporation from a free water surface in Central Arizona is about 6 ft/yr. Since the average infiltration for 1969 was about 220 ft, evaporation accounts for about a 3% salt increase. The remaining 1% may be attributed to solution of salt in the soil caused by a lowering of the pH due to CO_2-production by bacteria in the soil.

The salt content of WCW water began to drop in August 1969, indicating the start of the arrival of renovated water. The gradual nature of the reduction (Table 11) indicates that the renovated-water "front" has become diffuse after the long time and distance of underground travel.

The concentration of the main ions in the secondary effluent was as follows:

ion	ppm
HCO_3^-	381
Cl^-	213
SO_4^{--}	107
CO_3^{--}	0
Na^+	200
Ca^{++}	82
Mg^{++}	36
K^+	8
Total	1,027

The ionic composition of the renovated water can be expected to be about the same. The sodium adsorption ratio (SAR), defined as $Na^+ / \sqrt{(Ca^{++} + Mg^{++})/2}$ with the ionic concentrations in milliequivalents per liter, was about 4.6. This is well below the range

Table 11. Total salt concentration in effluent and in water from wells, mg/liter.

Date	Effl.	Renovated water						Native ground water		
		1	1-2	ECW	5-6	7	EW	WCW	8	91st Ave.
1968	800-992	1024-1182	909-1216	832-1158	1024-1088	1024-1152		2291-3712	2048-2176	
13-14 Jan 69	1056		1216	1088	1011			3800		
23 Jan 69		1152	1088	1024	960	1056			1984	
10-11 Feb 69	922		1120	1088	1024					
14-17 Feb 69	1088	1152	1088	1024	1056	1120			2048	
10-11 Mar 69	1024	1152	1152	1120	992	1152		3200	1856	2560
26 Mar 69		1056	960	992	1037	1216			1664	2432
14-15 Apr 69	979		1011	1024	1024					
28-29 Apr 69	1024		1216	1088	960			3600		
6 May 69	1088	1203	1152	1216	1120	1792			1677	2624
26-27 May 69	1024		1216	1184	1152					
6 Jun 69		1216	1280	1248	1248	1696		4000	1798	
30 Jun 69	966		1152	1210	1184	1613				
8-9 Jul 69	966	1152	1184	1184	1152	1408			1920	
28-29 Jul 69	960		1056	1024	1088	1120		3850		2560
5-6 Aug 69	973		1030	1024	1030	1050				
19-20 Aug 69	960	992	1043	1018	1082			3500	2048	
9-10 Sep 69	1152	1024	992	896	960	992			2048	
25-26 Sep 69	1152		1088	1152	1088			3150		
8-10 Oct 69	1216	1088	1056	1088	1056	1024			1948	2720
25-26 Oct 69	1056		1056	992	1024			2400		
14 Nov 69		992	845	928	896	1024			1888	
24 Nov 69	864		960	960	960		1152	2100		
5-8 Dec 69	960	960	1024	896	992	960	1024			2528
29-30 Dec 69	928		992	1043	947		960	1950	1856	
Av. 1969	10[illegible]		1082	1063	1043					

of 8 to 18 whereby damage to the structure of the soil can occur if the water is used for irrigation of soil containing clay. High values of SAR will cause the clay to deflocculate, which reduces the permeability of the soil and adversely affects the structure of the soil.

7. pH

The pH of the effluent was usually between 7.7 and 8.1, and of the renovated water between 7.0 and 7.6 (Table 12). The lower pH of the renovated water was probably due to bacterial activity in the soil and resulting CO_2-production. The pH of the native ground water ranged from 7 to 8 with an average of around 7.5.

8. Coliform and Soil Bacteria

Presumptive, confirmed, and fecal coliform densities were determined with the multiple-tube fermentation technique. Fecal coliform densities in the secondary effluent, which is not chlorinated, were usually of the order of 100,000 to 1,000,000 per 100 ml.

The fecal coliform density of the renovated water from ECW was generally less than 10 per 100 ml (Figures 24 and 25). If sequences of long inundation periods were held, the fecal coliform density in the renovated water tended to rise when newly infiltrated water arrived at the well, and then to decrease to essentially zero as inundation continued. The coliform removal was probably due to "filtering" at the surface and mortality in the hostile and competitive soil environment. Filtering and competition, and hence the fecal coliform removal, can be expected to increase with continued inundation, because of increased surface clogging and bacterial population in the soil. Later studies showed that essentially all the removal of fecal coliforms took place in the first 3 ft of the soil.

Fecal coliform densities in the renovated water from well 7 were lower than those from ECW, whereas fecal coliforms could not be detected in well 8 when it started to yield renovated sewage water. Thus, while a few feet of underground travel is sufficient to remove almost all fecal coliform bacteria, a distance of 100 to 200 ft is necessary to obtain renovated water with consistent absence of fecal coliforms.

Presumptive coliform densities were higher than the fecal coliform densities (Figures 24 and 25), indicating the presence of soil coliform bacteria such as Aerobacter aerogenes. The median density of fecal streptococci in the renovated water from ECW was 10/100 ml (Figure 25).

Plate counts of aerobic bacteria in the soil below the recharge basins indicated total populations of about 10^8 per gram of dry soil near the surface, and 10^6 to 10^7 per gram of dry soil at depths of about 3 ft (Table 13).

Table 12. pH of effluent and water from wells (1969).

Date	Effl.	Renovated water						Native ground water		
		1	1-2	ECW	5-6	7	EW	WCW	8	91st Ave.
1968	7.9-8.1	7.9	7.1	7.3-8.4	7.7	7.6		7.4-8.3	7.9	
13-14 Jan 69	7.24									
27 Jan 69	8.1	7.9	7.6	8.1	8.0	7.8		7.6	8.0	
20 Feb 69		7.8	7.2	7.2	7.7	7.4		7.9	7.7	
10-11 Mar 69	7.8		7.4	7.3	7.8			7.4		7.8
20 Apr 69								7.4		
6 May 69	7.7	7.6	7.1	7.1	7.8	7.5		7.4	7.7	
9-10 Jun 69	7.7	7.3				7.9			7.8	
26-27 Jun 69	8.1		7.1	6.9	7.6			7.3		
8-9 Jul 69	8.0	7.4	7.0	7.0	7.7	7.0		7.4	7.1	
22-23 Jul 69	7.6		7.3	7.2	7.7	7.4		7.2		7.5
4-5 Aug 69	8.0			7.1	7.6	7.2		7.1		
19-20 Aug 69	8.1	7.6	7.1	6.9	7.4			7.3	7.5	
9-10 Sep 69	8.1	7.7	7.2	7.1	7.4	7.2		7.3	7.5	
25-26 Sep 69	8.1		7.3	7.2	7.5					
8-10 Oct 69	8.2	7.9	7.6	7.4	7.7	7.4			7.6	7.4
23-24 Oct 69	7.8		7.1	7.0	7.3			7.3		
14 Nov 69		7.8	7.2	7.3	7.5	7.5		7.4	7.4	
24 Nov 69	8.1		7.2	7.3	7.4		7.4	7.4		
5-8 Dec 69	8.0	7.7	7.4	7.2	7.4	7.5	7.5	7.5		7.3
29-30 Dec 69	7.8		7.6	7.2	8.0		7.2	7.5	7.6	
Av. 1969	7.9		7.3	7.2	7.6					

Table 13. Aerobic bacteria counts in recharge basin soil.

Date	Basin	Depth inches	Population per gram of dry soil
2 July 1969	4 (grass)	0-1	9.2×10^7
	5 (rice)	0-1	3.7×10^8
21 July 1969	6 (rice)	0-1	4.2×10^8
	6 (rice)	12	4.3×10^8
	6 (rice)	36	4.0×10^6
21 July 1969	1 (bare area)	0-1	1.9×10^8
	1 (bare area)	36	2.2×10^7

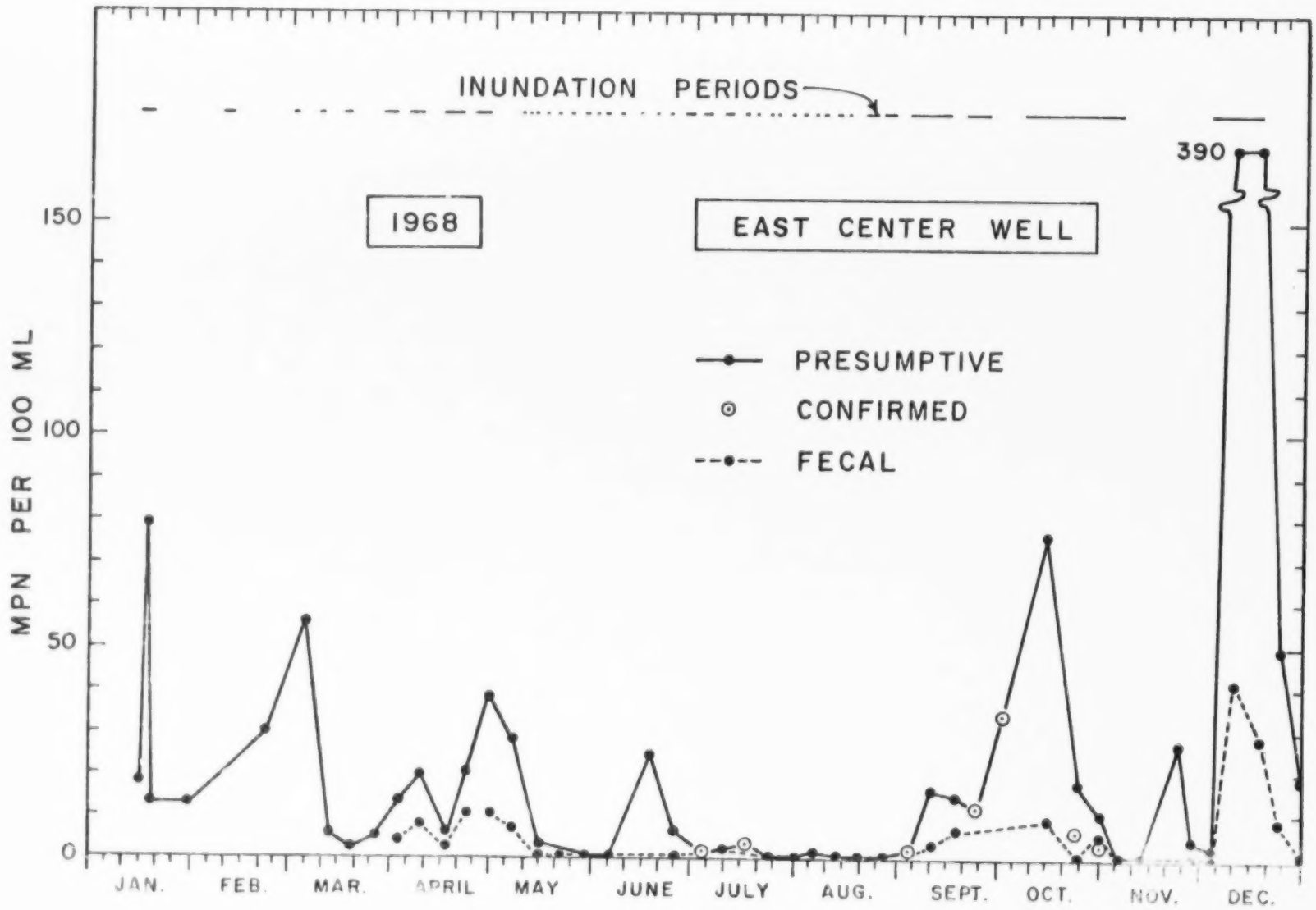

Figure 13. Coliform bacteria in renovated water from ECW in 1968.

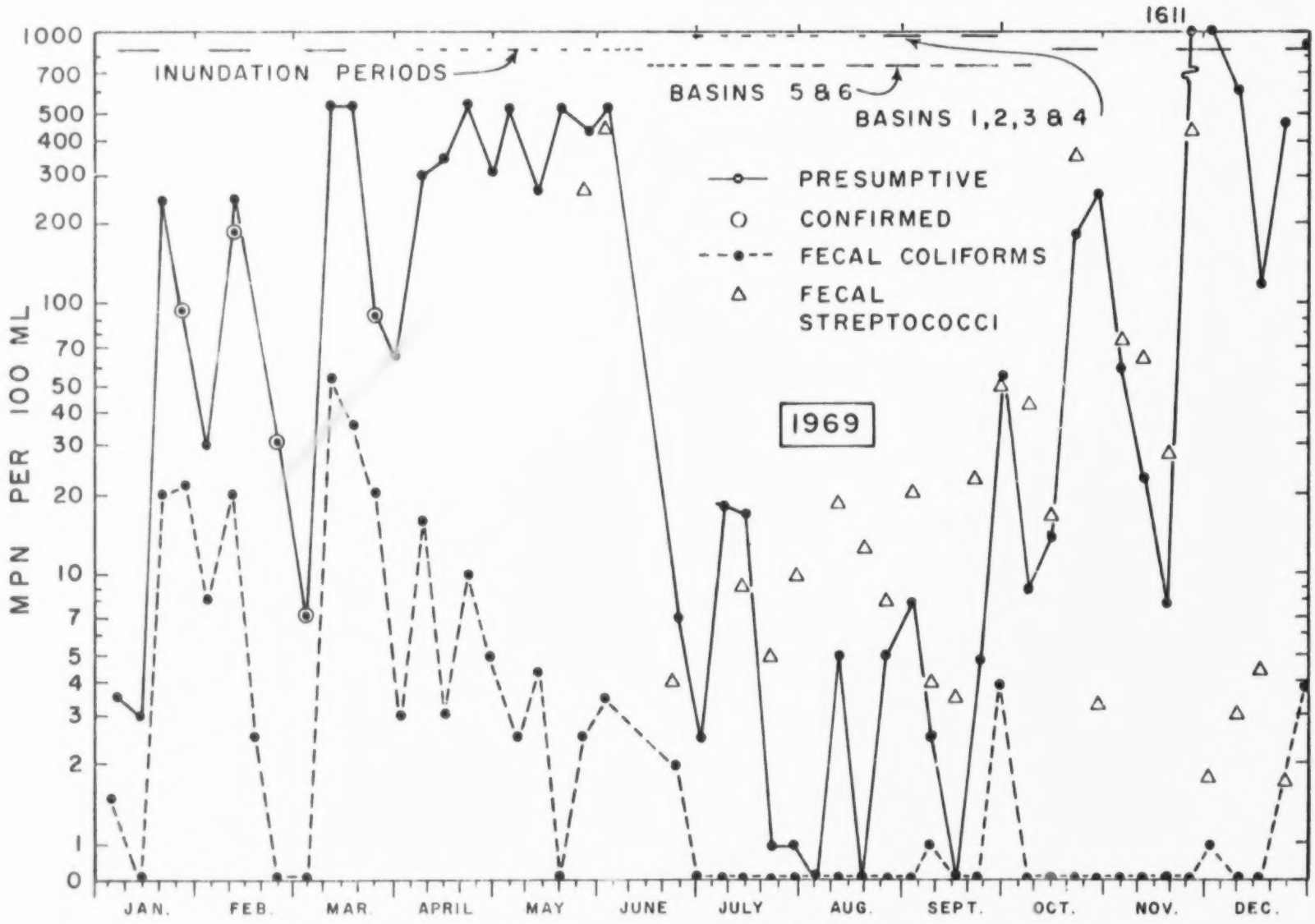

Figure 25. Coliforms and fecal streptococci in renovated water from ECW in 1969.

9. Effect of Grass Filtration on Effluent Quality

Passing waste water through grass or other vegetation as overland flow is sometimes successfully used as a treatment process. To investigate the effect of the flow through the grass basins on the quality of the effluent at the Flushing Meadows Project, the basins were split in two groups of three with the effluent flowing serpentine fashion through the three basins of each group (Figure 5). This "series" flow resulted in an overland flow distance of about 2100 ft through the grass in basins 4, 5, and 6.

Continuous samples of the effluent in the basins were obtained at flow distances of 0, 700, 1400, and 2100 ft, as indicated in Figure 5. Basin 1 had no vegetation and basin 2 was covered with a gravel layer. Thus, the flow for basins 1, 2, and 3 had passed through 700 ft of grass before entering the gravel basin No. 2, after which it entered the bare soil basin No. 1. The sampling devices consisted of a syphon tube which conducted water from the basin to a constant-head (overflow) cylinder. A long, small diameter tube, mounted on the cylinder wall near the top of the cylinder, conveyed water from the constant head device to a bottle at a slow enough rate to collect a sample of about 2 gallons in 24 hrs. The sample obtained of the effluent as it left basin 6 and entered basin 5 is referred to as sample "6-5." A similar notation was used for the other basins. The samples obtained at the outflow from basins 4 and 1 are referred to as "4-out" and "1-out." The grass filtration studies were carried out in the last half of 1968.

Since the grass was not mowed, a dense mat of about 6 inches thick and consisting of flat grass stems several feet in length had formed in basins 3, 4, 5, and 6 during the summer. Most of the overland flow of the effluent took place above this mat, through the green growth of the grass. However, the effluent that infiltrated into the soil had to move through the mat. To evaluate the quality of the effluent after it had moved through the grass mat, a perforated copper tube of about 3 ft length was placed in horizontal position on the soil surface below the mat. The location of this tube was near the flume connecting basin 6 to basin 5 and the sample is referred to as "6-5 tube." Continuous water samples from this tube were obtained using a device similar to that used in the basins.

The results of the analyses (Tables 14 and 15) show that, contrary to what might be expected, there was little effect of the overland flow through the grass basins on the quality of the effluent, even when the flow distance was about 2000 ft and the corresponding travel time 16 hrs. There seemed to be a slight tendency for the COD to increase as the effluent flowed through the grass, and for the ammonium and nitrate concentrations to decrease. The flow through the nonvegetated basins 1 and 2 also seemed to have little or no effect on the COD and ammonium content (Table 14).

Table 14. Effect of flow through basins on COD, NH_4, and NO_3 content of effluent (1968).

	COD, ppm O_2				NH_4, ppm N					NO_3, ppm N	
Sample	28-29 Aug	4-5 Sep	24-25 Sep	26-27 Sep	4-5 Sep	11-12 Sep	12-13 Sep	25-26 Sep	26-27 Sep*	28-29 Aug	12-13 Sep
Effluent (pump)	50	48	28	47	26.5	24.3	27.5	26.2	27.1	0.11	0.14
6-5	46	49	47	56	24.9	26.5	26.7	26.3	27.6	0.06	0.02
5-4	50	53	40	43	26.1	26.6	27.8	27.2	28.7	0.05	0.02
4-out	57	58	39	43	26.0	26.0	24.7	25.5	?	0.08	0.05
3-2	46	49	?	?	26.8	26.2	24.8	?	?	0.05	0.07
2-1	46	56	42	43	25.5	26.8	24.9	25.9	26.1	0.07	0.02
1-out	?	55	40	42	24.9	22.3	26.3	25.0	25.9	?	0.02

* Kjeldahl N

Table 15. Effect of flow through grass basins and grass mat on COD, NH_4 and NO_3 of effluent.

	14-15 Oct	15-16 Oct	22-23 Oct	29-31 Oct	3-4 Dec	4-5 Dec	5-6 Dec	6-9 Dec	10-11 Dec	11-12 Dec	12-13 Dec	13-16 Dec	16-17 Dec
COD, ppm O_2													
Effluent (pump)	43	38	41	25	68	63	?	?	22	43	?	61	49
6-5 tube	55	40	45	46	63	58	49	62	32	?	50	?	48
5-4	43	33	33	25	?	?	47	49	20	40	56	?	?
4-out	44	33	39	29	82	63	59	58	47	52	51	54	53
NH_4, ppm N													
Effluent (pump)	21.4		21.0	23.5	27.0	28.1	?	?	24.4	24.9	31.7	31.2	27.7
6-5 tube	20.8		17.7	17.3	29.1	28.9	28.0	29.6	19.5	?	26.4	?	27.9
5-4	20.4		17.8	19.3	?	?	29.4	28.6	39.3	35.0	30.5	?	?
4-out	18.8		17.5	17.1	32.2	28.9	30.8	30.5	34.6	29.5	31.6	34.3	28.5
Kjeldahl, ppm N													
Effluent (pump)			22.2	24.8	32.6	31.9	?	?					
6-5 tube			18.9	19.6	30.8	30.9	31.5	31.6					
5-4			19.2	21.0	?	?	32.4	31.8					
4-out			18.8	18.9	36.6	31.3	33.4	30.1					
NO_3, ppm N													
Effluent (pump)	?			0.33	0.0	0.2	?	?	0.2	0.00	0.02	0.05	0.02
6-5 tube	0.00			0.10	0.2	0.1	0.0	0.2	0.05	?	1.9	?	0.60
5-4	0.06			0.26	?	?	0.2	0.1	0.0	0.02	0.02	?	?
4-out	0.00			0.86	0.1	0.0	0.0	0.05	0.0	0.05	0.02	0.00	0.00

There was a tendency for the COD to increase as the effluent moved through the grass mat before it infiltrated into the soil (Table 15). This COD increase is probably due to release of organic matter from decaying grass stems and leaves. Also, there seemed to be a reduction in the ammonium content, probably because of nitrogen fixation by bacteria in the decaying mat. The nitrate content was not appreciably affected. The increase in COD as the effluent moved through the grass mat may have contributed to the increased denitrification below the vegetated basins, as discussed under the section "Nitrogen."

10. Water Temperatures

Temperatures of the secondary effluent and renovated water from ECW and WCW were measured in 1969. The temperature of the effluent at the inflow end of the basins ranged from 20°C in the winter to 32°C in the summer. The effluent temperature at the outflow end ranged between 13 and 30°C. The renovated water from ECW had temperatures of about 18°C in winter and 30°C in summer. For WCW-water, the temperature range was 21 to 28°C.

SECTION VIII

DESIGN AND OPERATION OF LARGE-SCALE SYSTEM

1. General Design Aspects

The infiltration studies of the Flushing Meadows Project have indicated that 1 acre of recharge basin can infiltrate at least 300 acre-feet of secondary effluent per year. Thus, reclaiming the present effluent flow of about 90,000 acre-feet per year would require about 300 acres of recharge basins. Renovating the sewage flow of 300,000 acre-feet per year projected for the year 2000 would require about 1000 acres of recharge basins. The recharge basins would be fairly large (about 300 x 500 ft each). They would probably best be located on both sides of the Salt River bed, one chain of recharge basins on the north side of the river bed, and another chain on the south side (Figure 26). The effluent would be distributed into the basins by a channel which would run along one side of the recharge strip.

After infiltration and lateral movement below the water table, the sewage water would be pumped as renovated water from a series of wells, which would be located in the center of the river bed (Figure 26). Since the river bed is about 2000 ft wide, the recharge strip could be about 500 ft wide, leaving a distance of about 500 ft between the recharge strips and the wells to insure adequate time and distance of underground travel.

Since a low suspended solids content in the effluent is required to obtain high infiltration rates with minimum maintenance of the basins, a sedimentation reservoir should be placed at the head of the channel system carrying the effluent to the recharge basins. Such a reservoir could also serve as a storage facility to even out diurnal fluctuations in the effluent discharge.

The design of a system for renovating sewage effluent by ground water recharge with infiltration basins should be based on the following criteria:

a. The water table below the recharge basins should be at a depth of at least 5 to 10 ft during infiltration. This is necessary to maintain an unsaturated zone of sufficient thickness for aerobic decomposition of organic material, and to permit rapid drainage, and hence aeration, of the soil profile between inundations.

b. The renovated water should have a certain minimum time and distance of underground travel before it is pumped from the well. This is necessary for "polishing" treatment, taste and odor removal, and for precaution against breakthrough of pathogenic organisms. While reliable data regarding desired time and distances of underground travel are lacking, an underground detention time of a few weeks and travel distances of a few hundred feet may be sufficient, depending on the aquifer

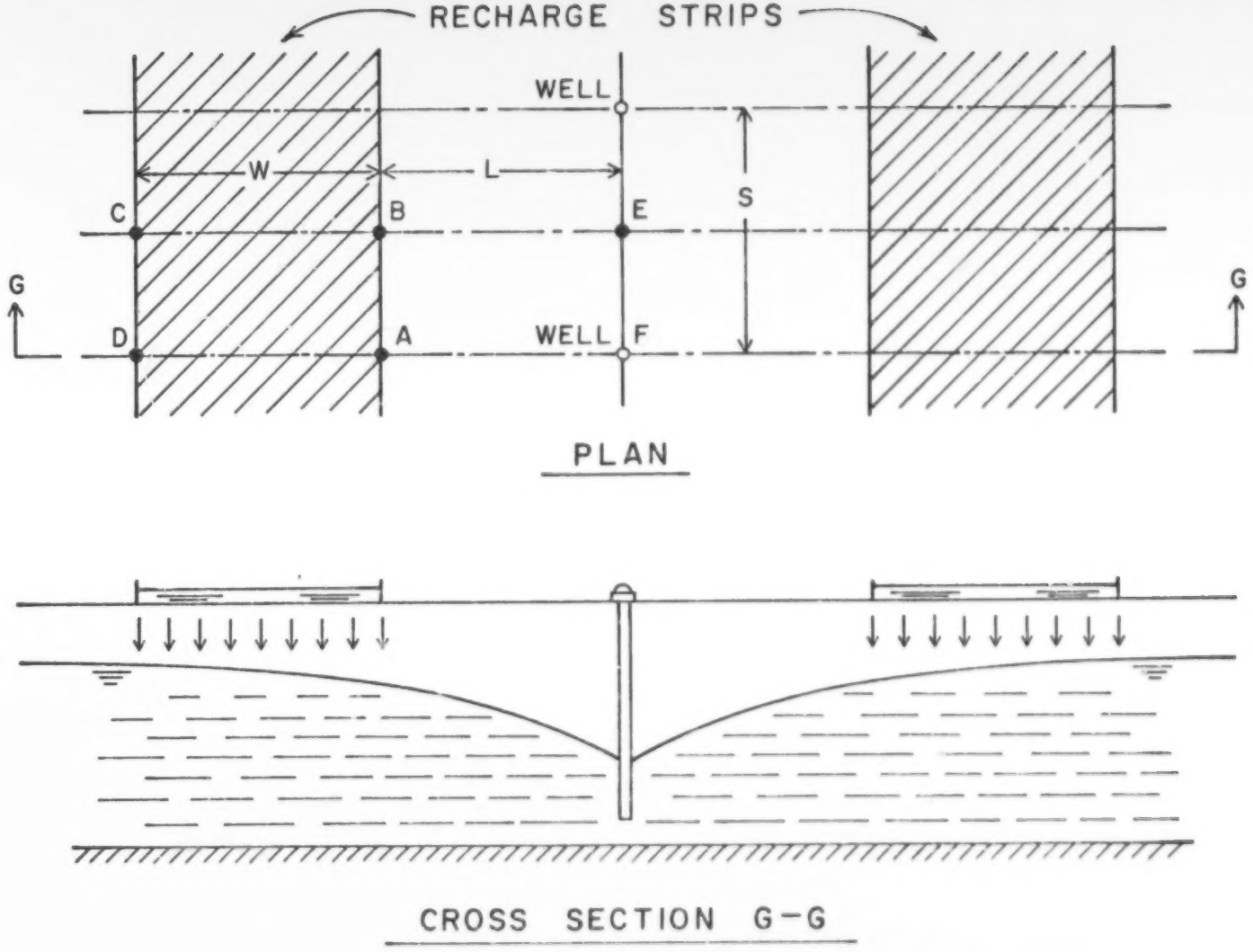

Figure 26. Plan and cross-section of two parallel strips with recharge basins with wells in center for collecting renovated water.

material, the quality of the effluent, and the desired quality of the renovated water.

c. The spread of the renovated water into the aquifer outside the system of recharge basins and wells should be avoided because the chemical composition of the effluent and the various processes in the soil are not completely known. Some compounds (hormones, enzymes, biocides, etc.) may resist biodegradation in the soil and if future research should show that some of these refractory compounds are toxic or carcinogenic, a difficult situation would exist if the renovated water had been allowed to spread uncontrolled into the aquifer. Thus, the system of Figure 26 should be operated so that all the effluent water that has infiltrated into the soil will be pumped from the wells in the center of the river bed. Also, the water table below the edges of the river bed (such as at points C and D in Figure 26) should not rise above the ground water level adjacent to the river bed. As a matter of fact, the ground water levels at points C and D should remain slightly lower than the water table outside the recharge system, so that there is a slight gradient from the aquifer to the recharge system to make sure that renovated sewage water cannot spread into the ground water outside the system.

2. Underground Flow System

In order to design a system in accordance with the above criteria, it must be possible to calculate water table positions and underground detention times that can be expected in the recharge system. For this purpose, analyses of the underground flow system for various system geometries and aquifer conditions were carried out with an electrical resistance network analog. Horizontal models of the flow system were simulated on the analog. This required knowledge of the effective transmissibility of the aquifer for ground water recharge.

The effective transmissibility, T_e, for recharge is less than the transmissibility of the entire aquifer because the recharge flow system is concentrated in the upper portion of the aquifer, often called the "active" region. The deeper portion of the aquifer, which does not contribute much to the recharge flow system, is called the passive region.

The effective transmissibility of the aquifer beneath the Salt River bed was evaluated by applying the Dupuit-Forchheimer assumption of horizontal flow to the recharge flow system of Figure 16. The shape of the ground water mound in this system is shown in Figure 27 with the vertical scale exaggerated.

According to the Dupuit-Forchheimer assumption, the horizontal flow beneath the ground water mound at a certain distance from the center can be described as

$$\bar{I}x = - T_e \frac{dh}{dx} \qquad (1)$$

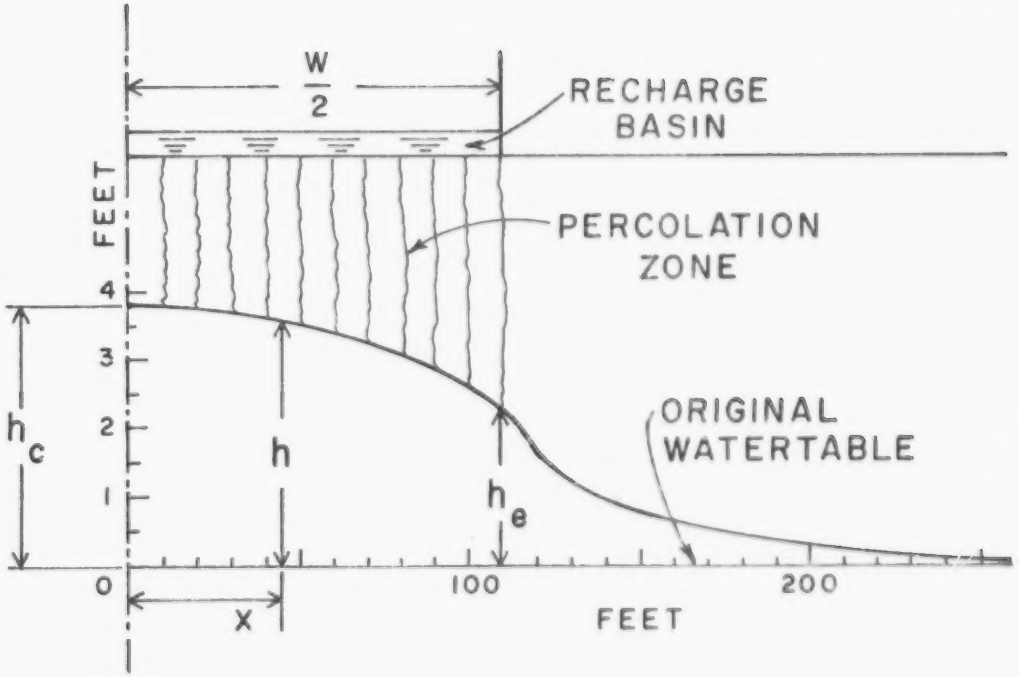

Figure 27. Ground water mound of flow system in Figure 16 with vertical scale exaggerated.

where $\bar{I}$ = infiltration rate for recharge basin area (ft/day)

T_e = effective transmissibility of aquifer (ft^2/day)

h = height of ground water mound above static water table

x = horizontal distance from symmetry line (Figure 27).

The factor $\bar{I}$ represents the average infiltration rate for the entire basin area, including the dry portions, such as dikes, roads, and basins undergoing a dry-up period. Of course, the inundated basins should be sufficiently frequent to permit treatment of the basin area as one infiltration unit with an average infiltration rate, $\bar{I}$.

Integrating equation (1) for the region beneath the recharge area yields

$$h_c - h_e = \frac{\bar{I} W^2}{8 T_e} \qquad (2)$$

where h_c = h at center of mound

h_e = h at edge of mound (x = W/2)

W = width of infiltration area (Figure 27).

For the ground water mound in Figure 27, h_c = 3.80 ft, h_e = 2.35 ft, $\bar{I}$ = 1.91 ft/day, and W = 220 ft. Substituting these values in equation (2) yields a value of 7,970 ft^2/day, rounded to 8000 ft^2/day, for the effective transmissibility of the aquifer. Since the hydraulic conductivity in horizontal direction is 282 ft/day, the effective height of the flow system in Figure 16 is about 28 ft. According to Figure 16, this depth corresponds approximately to the 60% streamline. The total height of the unconfined aquifer was much greater, i. e., about 240 ft (Table 1).

The effective transmissibility for ground water recharge may be a constant, or it may depend on the width of the recharge basin area. If the transmissibility of the aquifer is mainly due to a single layer of very high hydraulic conductivity, the effective transmissibility will be essentially constant. If, however, the aquifer is uniform, or uniformly anisotropic, to appreciable depth, the effective transmissibility will vary in direct proportion to the width of the recharge basin area, until the effective height of the flow system is about equal to the height of the aquifer (vertical distance between water table and impermeable boundary).

For the unconfined aquifer beneath the Salt River bed, with its sand and gravel layers at great depth, the effective transmissibility should be taken in direct proportion to the width of the recharge basin area.

If the hydraulic properties of the aquifer are similar to those at the Flushing Meadows site, the effective transmissibility for systems of different widths can be calculated as

$$T_e = \frac{W}{220} \; 8000 \; ft^2/day \qquad (3)$$

where W is the width of the basin area or recharge "strip" for which the effective transmissibility is to be calculated. The factors 8000 and 220 in this equation refer to T_e in ft^2/day and W in ft, respectively, of the Flushing Meadows Project.

Analyses of the flow system for the large-scale system (Figure 26) were performed for different geometries of recharge strips and wells and different effective transmissibilities. The analyses were based on steady-state conditions. Thus, all the water that infiltrated in the basins was assumed to be pumped from the wells and the well discharge could be calculated as

$$Q_w = 2 \; SW\bar{I} \qquad (4)$$

in which Q_w = discharge per well

S = distance between wells

W = width of recharge strip

$\bar{I}$ = average infiltration rate for recharge strip (Figure 26).

Because of the symmetry of the system in Figure 26, only one quadrant of the flow system around each well, such as section CDEF, needed to be analyzed. The analysis was based on the assumption that the water table below the outside edges of the recharge strip (CD in Figure 26) would be maintained at the same elevation as the water table in the aquifer outside the recharge system, so that there would be no flow from the recharge system to the outside aquifer, or vice versa.

Assuming that the ground water flow beneath the recharge strip is rectilinear and parallel to CB (Figure 28), the elevation difference, ΔH_{C-B}, between the water table at C and B could be calculated with equation (2) as

$$\Delta H_{C-B} = \frac{\bar{I} \; W^2}{2 \; T_e} \qquad (5)$$

Because there is no symmetry line in the center of the recharge strip for the system of Figure 26, the factor 8 in the numerator of equation (2) was replaced by a factor 2 in equation (5). The water table difference, ΔH_{D-A}, between D and A is equal to ΔH_{C-B}, because of the assumption of parallel flow in section ABCD.

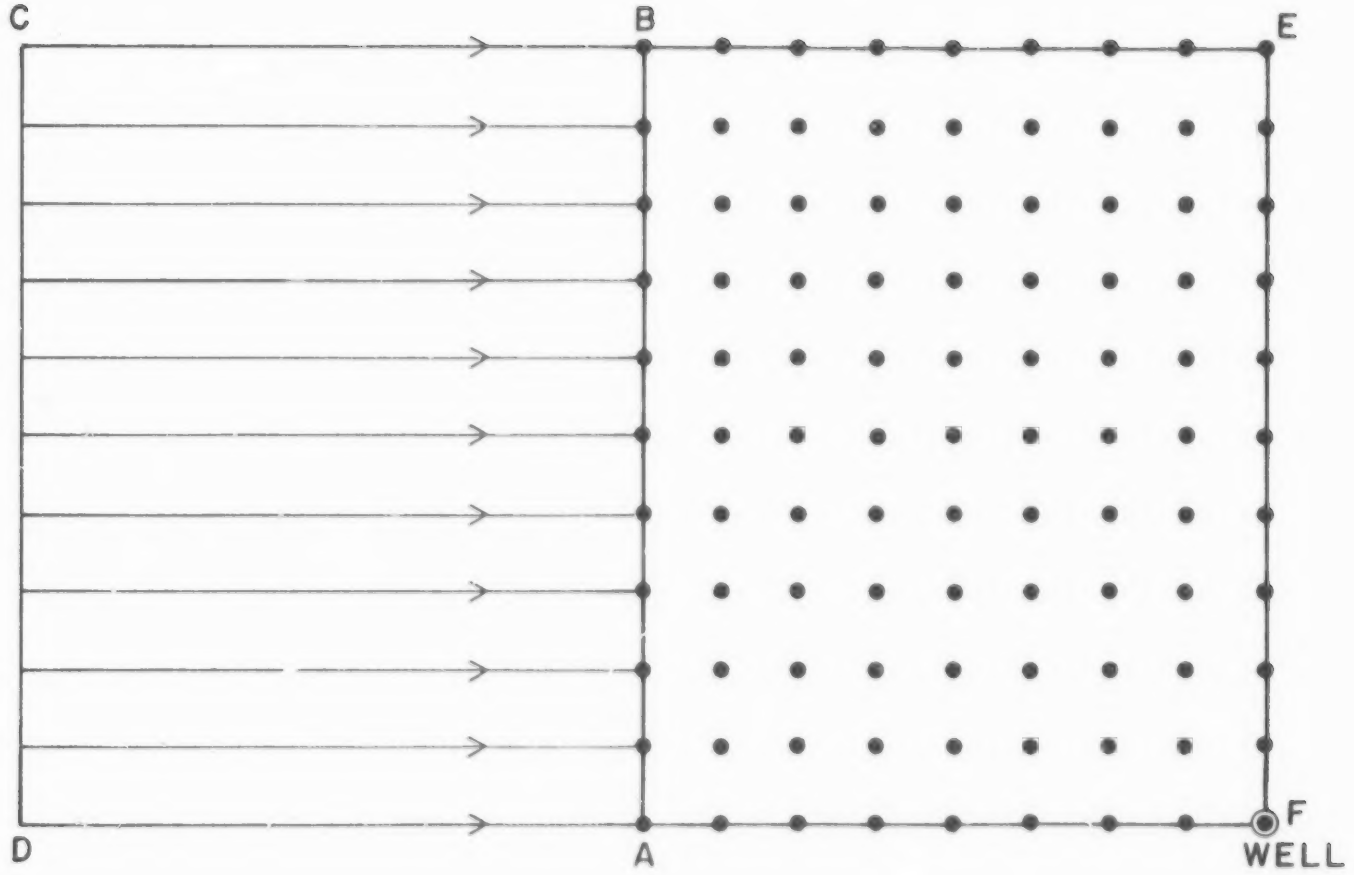

Figure 28. Linear flow beneath recharge strip (ABCD) and node arrangement in portion of flow system (ABEF) analyzed by resistance network analog.

To determine the elevation difference between the water table adjacent to the well (point F in Figure 26) and the water tables at A and B, the section ABEF was simulated on a resistance network analog. The line AB was taken as a line source with uniform flux and the well as a point sink. The analog analyses were performed for different ratios of L/S, in which L = distance between recharge area and well, and S = distance between wells (Figure 26). The node arrangement for the case where L/S = 0.5 is indicated in Figure 28. The results of the analog analyses are shown in dimensionless form in Figure 29, where the water table elevation drop, ΔH_{A-F}, from A to F is plotted versus L/S in terms of the dimensionless parameter $2\,WS\bar{I}/T_e\,\Delta H_{A-F}$. Another dimensionless graph (Figure 30) shows the ratio $\Delta H_{B-F}/\Delta H_{A-F}$ versus L/S. As long as L/S > 0.5, this ratio is essentially equal to 1, so that the water table elevations at A and B are nearly the same. Thus, the assumption of one-directional flow in section ABCD is valid if L/S > 0.5.

Equation (5) and Figure 29 can be used to calculate the elevation difference between the water table at the wells (point F) and the water table beneath the outer edges of the recharge strips (points C and D). This elevation difference can be used to determine the dynamic lift for pumping the renovated water from the wells, adapting the water table below the outer edges of the infiltration strips (C and D, Figure 26) to the original water table.

Since the well was simulated as a point sink in the analog analyses, the actual diameter of the well was not taken into account and the results are approximate only. To adapt the results to a well of given diameter, the theory of steady, radial flow to a well (see, for example, Todd (12), Chapter 4) can be applied to the region around the well where the equipotentials are essentially circles (Figure 32).

The underground detention time consists of time for downward travel to the ground water and time for lateral travel to the well. The downward travel time can be calculated from the infiltration rate, the soil porosity and water content in the percolation zone, and the distance of vertical travel. The horizontal travel time depends on the distance from the point of infiltration to the well. The water that has infiltrated at A will be of most interest, because it will have the minimum horizontal travel time occurring in the system (Figure 26).

To evaluate the minimum underground travel time, equipotentials and streamlines in the region ABEF must be known. An example of the equipotentials and streamlines in this region obtained by electrical resistance network analog is presented in Figure 31. To predict the travel time from A to F in this system, the flow rate in the stream tube between AF and the next streamline is calculated by multiplying the width of the tube along AB by $\bar{I}W$. This flow rate is then divided by the cross-sectional area of the tube (width of tube x height of

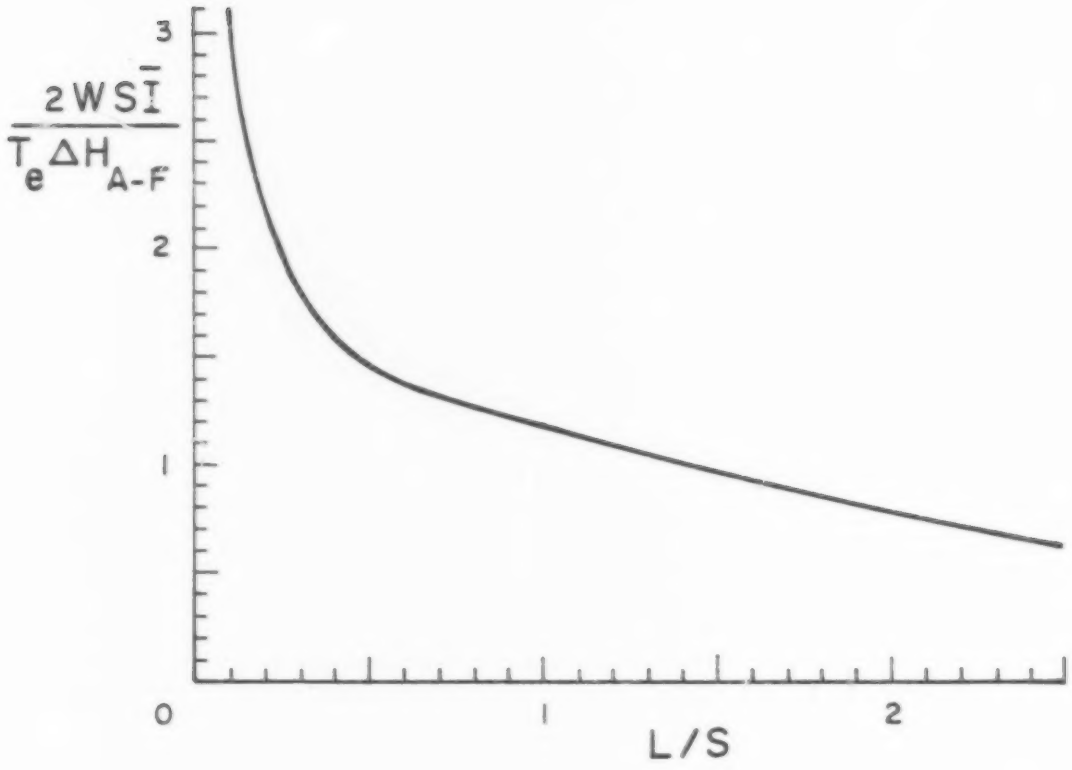

Figure 29. Dimensionless graph of $2WS\bar{I}/T_e \Delta H_{A-F}$.

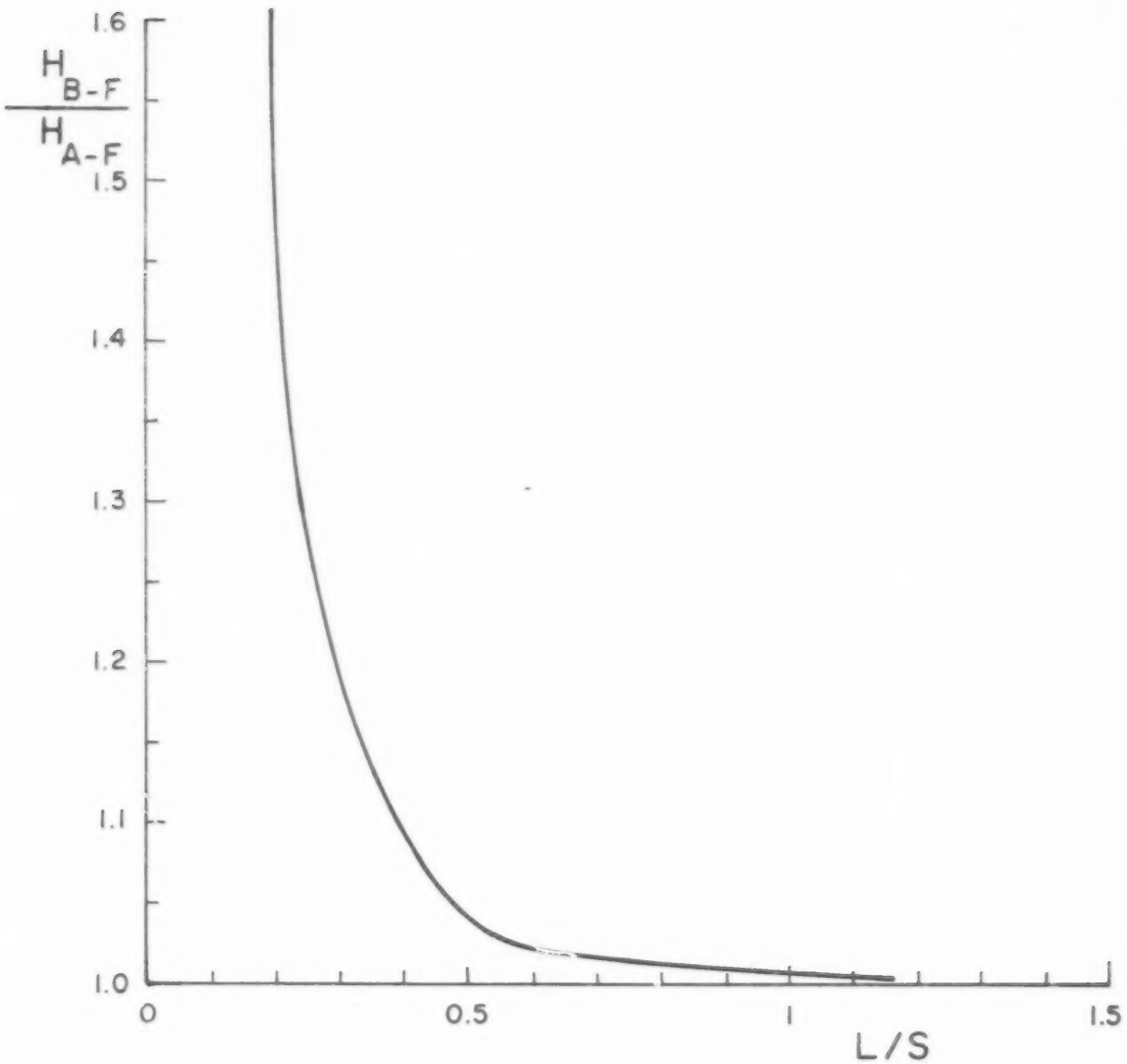

Figure 30. Dimensionless graph of $\Delta H_{B-F}/\Delta H_{A-F}$ versus L/S.

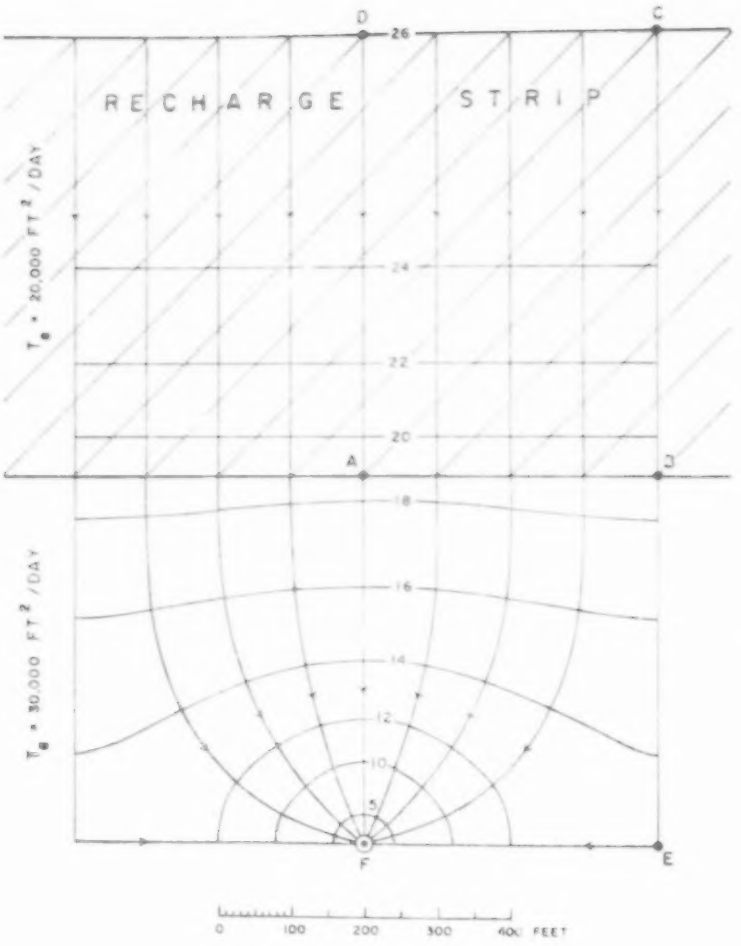

Figure 31. Streamlines and equipotentials (in feet above water table adjacent to well) for hypothetical system in Salt River bed.

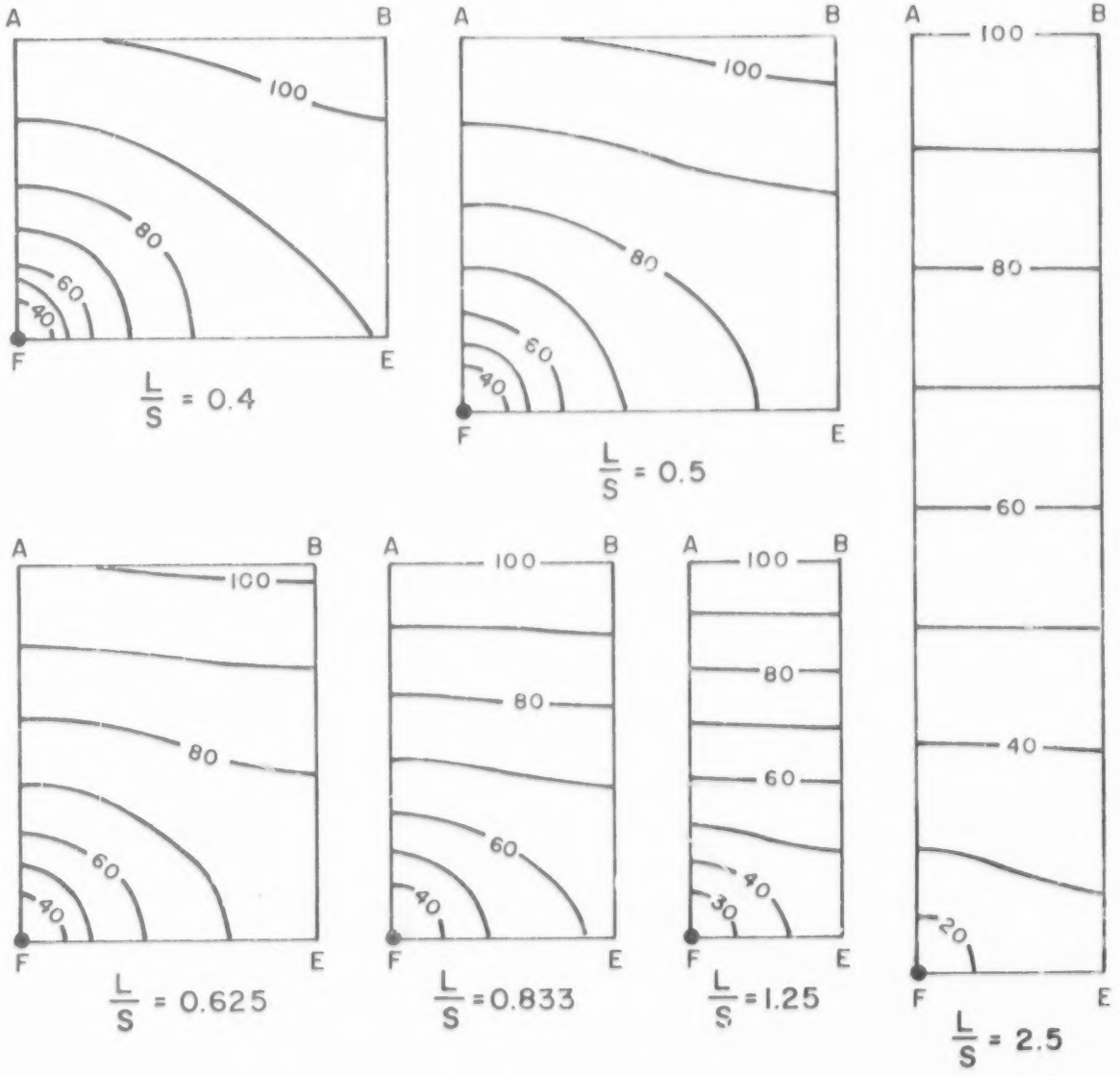

Figure 32. Equipotentials (AB = 100, F = 0) in region ABEF for different values of L/S.

aquifer) and by the porosity of the aquifer material to yield the macroscopic velocity. This velocity can be computed for each section of the stream tube between successive equipotentials. Dividing the distance between these equipotentials by the macroscopic velocity for the section in question then yields the travel time increment for that section. The total travel time from A to F is obtained by summing the time increments for each section in the stream tube.

To enable calculation of travel times for different system geometries, equipotentials for the the region ABEF were determined for different ratios of L/S (Figure 32). These equipotentials are expressed in percent head loss between A and F and they were evaluated by resistance network analog using node arrangements similar to the one shown in Figure 28. Streamlines can be sketched as orthogonals to the equipotentials, as was done in Figure 31.

The effective transmissibility, T_e, for the flow region near the well may be greater than T_e for the flow region below the recharge area. This is true if the aquifer is unconfined and the wells penetrate the aquifer to a greater depth than the height of the active region of the aquifer for the recharge flow system. In that case, it will be desirable to treat the flow in the region ABDF (well-flow region) with a higher T_e-value than the one for the recharge flow in region ABCD. This is permissible, since most of the head loss between the recharge area and the well occurs in the vicinity of the well (Figures 31 and 32). The value of T_e for the well-flow system would then be evaluated on the basis of the well depth or of pumping tests, whereas the value of T_e for the recharge-flow system would be based on equation (2), corrected for the width of the recharge area by equation (3) if necessary.

Example. To illustrate the use of equation (5) and Figures 29, 30, and 32, the height of the water table at A, B, C, and D (Figure 26) above that of the water table adjacent to the well, and the travel time from A to F, will be calculated for a hypothetical system in the Salt River bed.

Taking W = 600 ft, T_e for the flow system beneath the recharge basins (ABCD, Figure 26) can be calculated with equation (3) as about 21,850 ft^2/day, which will be rounded to 20,000 ft^2/day. Because the wells will probably penetrate the aquifer more than the height of the active zone for the recharge flow system, T_e for region ABEF will be taken as 30,000 ft^2/day.

The annual recharge rate will be taken as 300 ft, giving an $\bar{I}$-value of 0.82 ft/day which will be rounded to 0.8 ft/day. The pumping rate will be taken as 768,000 ft^3/day for each well, which corresponds to a well discharge of about 4000 gpm.

Substituting the values for Q_w, W, and $\bar{I}$ in equation (4) yields S = 800 ft. This gives L/S = 0.625, so that according to Figure 29,

$2\,WS\bar{I}/T_e\,\Delta H_{A-F} = 1.38$. Substituting the values for W, S, $\bar{I}$, and T_e for the well-flow system into this parameter yields $\Delta H_{A-F} = 18.6$ ft. Figure 30 shows that for L/S = 0.625, $\Delta H_{B-F}/\Delta H_{A-F} = 1.02$, so that $\Delta H_{B-F} = 18.9$ ft. The next step is to calculate ΔH_{C-B} with equation (5), which yields $\Delta H_{C-B} = 7.2$ ft. Thus, $\Delta H_{D-F} = 18.6 + 7.2 = 25.8$ ft, and $\Delta H_{C-F} = 18.9 + 7.2 = 26.1$ ft. The water table adjacent to the well is thus about 26 ft lower than the water table beneath the outside edges of the recharge strips.

Equipotentials and streamlines for this example are shown in Figure 31. The equipotentials for the recharge flow system in region ABCD were calculated with equation (5), and those for the well-flow system in section ABEF were obtained from Figure 32. The streamlines were sketched as orthogonals to the equipotentials. To calculate the minimum travel time, K_h will be taken as 282 ft per day (Figure 16) so that the effective height of the aquifer for the well-flow system is 30,000/282 = 106 ft. Assuming a porosity of 15%, the macroscopic velocity between the 18-ft and 16-ft equipotentials in the stream tube between AF and the next streamline can be estimated as (0.8 x 600 x 100)/(123 x 100 x 0.15) = 26 ft per day. In this calculation, the numerator is the product of recharge rate and recharge area feeding the stream tube, the factor 123 is the average height of the aquifer between the 18-ft and 16-ft equipotentials (106 + (16 + 18)/2), the factor 100 in the denominator is the average width of the stream tube between the 18-ft and 16-ft equipotentials, and the factor 0.15 represents the porosity. Dividing the average distance between the two equipotentials, which is 115 ft, by the macroscopic velocity, yields a travel time of 4.4 days. Applying this procedure to the rest of the potential intervals of the stream tube and summing the time increments yields a total travel time of 2 weeks from A to the well. If a longer minimum travel time is desired, S can be decreased, L increased, $\bar{I}$ decreased, and W decreased.

Prediction of systems of underground water movement is fraught with uncertainty where nonuniform hydrogeologic conditions exist. Thus, design calculations may have to be followed up by field measurements to determine how the actual performance of the system compares with the predictions, and what modifications in the operation of the system may be desirable. Such field studies should include measuring water levels in observation wells and determining the quality of the water as it moves to and below the water table. Travel times can be evaluated by following the movement of a characteristic ion or compound present in the waste water, or by adding a tracer.

3. Economic Aspects

The total cost of renovating sewage water with the operational system of Figure 26 is estimated at about $4.50 per acre-foot. This figure is calculated as follows:

Cost of land *	$ 2000 per acre
Construction (sedimentation reservoir, channels, distribution works, dikes, etc.)	2000 per acre
Wells ($30,000 per well, one well per 15 acres)	2000 per acre
Total capital cost	$ 6000 per acre

* Some land in the Salt River bed is in private ownership. The rest is state or federal land.

Annual fixed cost (10% of capital costs)	$ 600 per acre
Annual maintenance	150 per acre
Total annual cost	$ 750 per acre

With an annual infiltration rate of 300 ft, the above cost amounts to $2.50 per acre-foot. Assuming a pumping lift of 100 ft and pumping cost of 2 cents per acre-foot per foot of lift, the pumping costs would be $2.00 per acre-foot. Thus, the total cost of infiltrating the effluent and pumping the renovated water would be $4.50 per acre-foot.

A more detailed cost analysis of renovating sewage water with the system of Figure 26 was made by Buxton (8), who arrived at a total-cost range of $4.83 - $5.87 per acre-foot.

Because the hydrogeologic conditions of the Salt River bed and the climate of the lower Sonoran Desert are favorable for renovating sewage water by soil filtration and ground water recharge, renovated sewage water appears to be the cheapest and most readily available "new" water resource for Central Arizona. The cost of renovating the sewage water by in-plant tertiary treatment (phosphate precipitation, ammonium stripping or nitrification-denitrification, activated carbon adsorption, and disinfection) would be at least $50 per acre-foot, or about 10 times the cost of renovating the sewage water by ground water recharge.

4. Future Projects

Because of the favorable results from the Flushing Meadows Project, larger projects can be embarked upon with confidence. One such project would be at the Phoenix 23rd Avenue Sewage Treatment Plant, where a 40-acre oxidation pond could be split into four 10-acre infiltration basins. These basins would be intermittently flooded to infiltrate about 15 mgd of the secondary effluent (activated sludge). The renovated water could be pumped from wells in the center of the basin area, and delivered to an irrigation-district canal for unrestricted irrigation.

Another project involving use of renovated sewage water is the Rio Salado Project, proposed in 1966 by Arizona State University's Architecture Department. This project would consist of converting the Salt River

bed into aquifer parks, housing and industrial developments, etc. Part of the river bed would be used for infiltration basins. The renovated water would be pumped upstream to be cycled through chains of recreational lakes. The remainder of the renovated water could be pumped into the canal system for unrestricted irrigation. Some of this water could also be used for ground water recharge where the ground water table is low because of too much pumping. A model of the proposed system, showing the waste water renovation system and the recreational lakes, is shown in Figure 33.

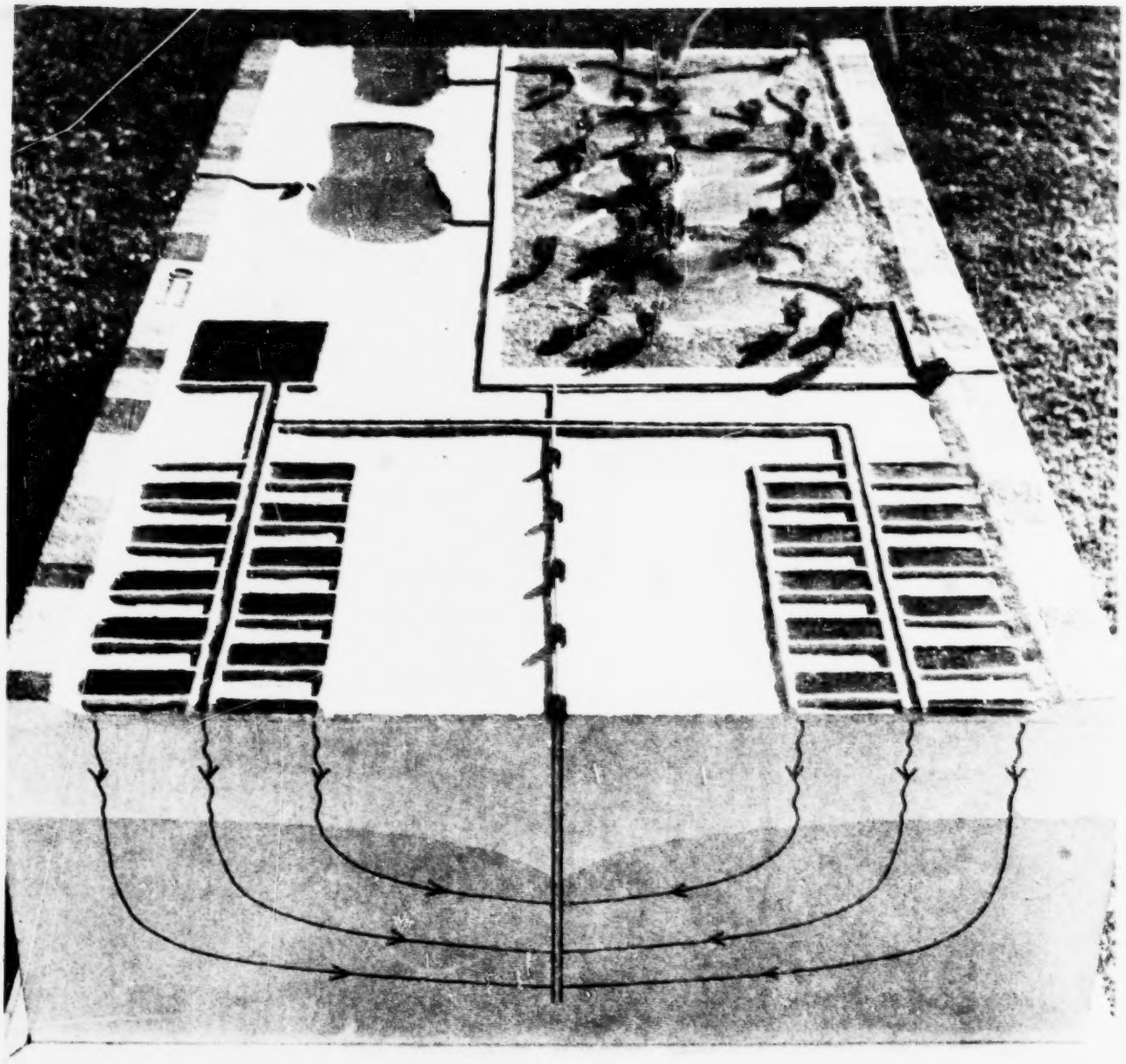

Figure 33. Model of Rio Salado Project, showing sewage water renovation system (front) and recreational lakes (back).

SECTION IX

ACKNOWLEDGMENTS

Meetings between various agencies in the Salt River Valley to discuss means for upgrading and reusing the valley's municipal waste water were held in the early 1960's. Participating in these meetings were, among others, H. Shipley (Salt River Project), D. Travaini (City of Phoenix), and L. E. Myers (U. S. Water Conservation Laboratory). Plans for a pilot project to study renovation of the effluent by ground water recharge were developed in 1964 by Herman Bouwer (U. S. Water Conservation Laboratory), and an Effluent Recharge Committee was formed to carry the project to reality. The committee consisted of:

Herman Bouwer, U. S. Water Conservation Laboratory (Chairman)
C. A. Pugh, Bureau of Reclamation
H. Shipley, Salt River Project
D. Travaini, City of Phoenix
J. J. Weinstein, Maricopa County Health Department
L. G. Wilson, University of Arizona

Numerous meetings were held and an application for a demonstration grant from the Environmental Protection Agency (then Federal Water Pollution Control Administration) was forwarded by the Salt River Project in March 1966. The grant was approved in December 1966 and construction began in early 1967 by the Salt River Project. H. Shipley, Associate General Manager of the Salt River Project, was the project officer for the grant. The research program, which was carried out by the U. S. Water Conservation Laboratory, started in September 1967.

Many persons have contributed to the project. The "effluent team" of the U. S. Water Conservation Laboratory consisted of Herman Bouwer, R. C. Rice, and E. D. Escarcega. It was later joined by J. C. Lance and F. D. Whisler. The water analyses were done, in succession, by John Krebs, Paul Kuechelmann, and M. S. Riggs of the Salt River Project, stationed at the U. S. Water Conservation Laboratory. F. S. Nakayama of the U. S. Water Conservation Laboratory developed the initial procedures for the water analyses. J. A. Replogle of the U. S. Water Conservation Laboratory designed the flumes for measuring flow into and out of the infiltration basins. Of the Salt River Project, H. Shipley, R. W. Teeples, G. Garrison, O. Hatcher, R. L. Juetten, D. E. Womack, R. Pristo, and W. L. Simser rendered valuable services to the development and construction of the project. Of the City of Phoenix, the cooperation of Dario Travaini, E. H. Braatelien and A. E. Watson is acknowledged.

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8. Performing Organization Report No.

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16. Abstract

A field project demonstrated the feasibility of renovating secondary sewage effluent by ground water recharge with infiltration basins. Maximum loading rates were obtained with cycles of 20 days flooding rotated with dry periods of 10 days in the summer and 20 days in winter. With these schedules the system could infiltrate 300-400 ft/year using a water depth of 1 ft. Grassed basins had higher infiltration rates, and a gravel covered basin had a lower infiltration rate than a bare soil basin. Essentially complete removal of BOD and fecal coliform, and significant removal of phosphorus, nitrogen and fluoride were obtained. Hydraulic properties of the aquifer were evaluated by analog from the response of piezometric heads in the ground-water system to infiltration. These properties were then used in the design of a prototype system, which would yield renovated water at an estimated total cost of about $5 per acre-foot at the pump.

17a. Descriptors

*sewage disposal, *tertiary treatment, *water reuse, *ground water recharge, *water spreading, aquifer characteristics, water table, design, infiltration, oxygen demand, nitrogen, denitrification, phosphates, fluorides, boron, coliforms

17b. Identifiers

Flushing Meadows Project, Salt River bed, Phoenix, Arizona

17c. COWRR Field & Group

18. Availability

19. Security Class. (Report)

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